VOL.II CHAPTER 4
METEORO-HYDROLOGICAL SURVEY

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4.1 Discharge at the Scheme Sites

(1) Application of Discharge Data

As a result of study of discharge data, gauging station records to be applied to the micro-hydropower generation plan are as follows.

1) Adardour and Inzaine Schemes

Gauging stations are located at Iquir N'kouris and Imim el Hammam at the upstream of N'fis river; however, analysis of the rainfall isohyetal map indicates precipitation considerably less than that occurring in the Project area. Accordingly, it was determined to apply discharge data from Tahanaout gauging station, the next closest station to the area.

2) Alla Oumzri and Id Ssior Schemes

These schemes make use of spring discharge, and according to local residents this shows no major fluctuation throughout the year. As this discharge is currently used for irrigation, discharge observations were carried out at the irrigation canal. Results of observation indicate discharge in the dry season of approximately 0.1 m^3 in both cases.

- Alla Oumzri: September 9, 1996 0.096 m^3/s
- Id Ssior: September 10, 196 0.100 m^3/s

3) Tidsi Scheme

The Taferiat gauging station is located downstream of the scheme area; however, records thereat show a large difference with annual rainfall in the actual scheme area. Furthermore, catchment area ratio for the station is small, and records are considered to be lacking in reliability. Accordingly, discharge data from the Aghbalou gauging station was applied to this scheme.

4) Arg and Anfli Schemes

Discharge records from stations downstream of these scheme areas, i.e. Tahanaout and Aghbalou, respectively, were applied to the said schemes.

5) Discharge observations at the newly established gauging stations under the Study are discussed in Section 4.3; however, these data were not applied to the subject scheme planning in light of the fact that water level and discharge observations were carried out only over a very limited 9 month period.

(2) Discharge at the Scheme Sites

Discharge data from the Tahanaout and Aghbalou gauging stations only was applied to the generation planning under this Study.

On the basis of annual rainfall, catchment area size, specific discharge and runoff coefficient, discharge data from Tahanaout station was applied to schemes such as Adardour, Inzaine and Arg where specific discharge is relatively less and catchment runoff coefficient large. Conversely, discharge data from Aghbalou station was applied to schemes such as Anfli and Tidsi where specific discharge is relatively greater and catchment runoff coefficient small.

As indicated below, discharges for Adardour, Inzaine and Arg were computed based on the discharge observations at Tahanout applying a conversion factor taking into consideration catchment area size and annual rainfall amounts. Discharges for Anfli and Tidsi were computed in the same manner based on data at Aghbalou. Results of the foregoing are indicated in Table 4.1-2.

Conversion Factors for the 5 Micro-hydropower Sites

Site	Tahanaout	Adardour	Inzaine	Arg	Aghbalou	Anfli	Tidsi
Area (km^2) Ai	A1=226	23	79	48	A1=503	134	24
Rainfall (mm) Ri	R1=450	600	600	650	R1=550	700	700
Conversion factor		0.14	047	0.31		0.34	0.06

(note: annual rainfall based on isohyetal map; conversion factor (Ai/A1) × (Ri/R1))

Diagram representation of the daily discharges for the scheme sites based on the above is given in Figure 4.1.1~4.1-5.

Discharge characteristics at the scheme sites are indicated in Figure 4.1-6~4.1-10, and discharge values are shown in Table 4.1-3.

Table 4.1-1 Gauging Stations Applied to the Schemes

Tributary	Scheme	CA (km^2)	Existing gauging station	CA (km^2)	New gauging station	CA (km^2)	Station applied
Amizmiz	Adardour	23	Iquir N'kouris	848	Infag	79	Tahanaout
Amizmiz	Inzaine	79	Iquir N'kouris	848	Infag	79	Tahanaout
Rhenaya	Arg	48	Tahanaout	226	Agr	48	Tahanaout
N'fis	Alla Oumzri	8	Iguir N'kouris	848	--	--	Spring
N'fis	Id Ssior	7	Iguir N'kouris	848	--	--	Spring
Ourika	Anfli	134	Aghbalou	503	--	--	Aghbalou
Zat	Tidsi	24	Taferiat	515	Tidsi	24	Aghbalou

4.2 Installation of Gauging Stations

(1) Selection of Gauging Stations

The candidate villages selected by Haouz Province for electrification by micro-hydropower are located within the catchment areas of the 4 major tributaries of the Tensift river, i.e. the N'fis, Rheraya, Ourika and Zat rivers. The subject micro-hydropower scheme sites are located in the basins of branch tributaries of these 4 rivers, i.e. the Imenane, Anougal, Eldouz, Ansa, Afoughal, Tighdouine and Yagour rivers. Of these, the majority of scheme sites are concentrated in the Imenane, Ourika and Anougal basins.

In determining the necessary discharge at each river to support the envisioned micro-hydropower generation plan, it would be ideal to have access to long-term discharge observation data from a point near the sites. However, such observation data for the most part is lacking. Fortunately, there are 5 existing gauging stations at the mid reaches of the 4 major tributaries in question for which long-term records are available. On the other hand, nevertheless, the micro-hydropower candidate sites under the Study are located for the most part along mountain torrents at points with small catchment areas and river gradient of 1/15~1/40. Existing observational data on discharge for these sites is unavailable. Although discharge can be estimated on the basis of rainfall runoff analysis, precipitation data as well is lacking. Accordingly, the following 3 sites were selected on the basis of good existing access roads as locations for new gauging station installation.

Site Features for New Gauging Station Installation

Gauging station	Infag	Arg	Tidsi
River	Anougal	Imenane	Afoughal
Catchment area (km²)	79	48	24
Latitude (° - ' - " N)	31-04-31	31-11-17	31-20-33
Longitude (° - ' - " W)	8-16-50	7-55-19	7-26-29
Elevation (m)	1,250	1,480	1,750

(2) Installation of Gauging Stations

Three gauging station sites were selected on the basis of field reconnaissance, and observational equipment installed thereat. The purpose of the installed equipment is to render observations on discharge and water level in the vicinity of the micro-hydropower scheme sites. One set of water level marker was installed at each gauging station site; however, under this Study these were not of the automatic recording type. Installation works were contracted locally to Maghreb Projets A.C., and the subject works were completed for all sites on July 7, 1996. Figure 4.2-1 gives the river cross-section at each of the new gauging station sites.

4.3 Discharge and Water Level Observations

(1) Discharge Observations

The period of discharge observation comprised 1 year from July 1996 to June 1997. During the actual Study period, observations were carried out at a rate of around 1 time per month for a total of 10 times. Due to impeded access during the winter season, observations were not carried out in December 1996 and January 1997. The purpose of discharge observation was to evolve water level ~ discharge curves for the schemes and correlate discharge on the basis of water level measurement. Discharge measurements works as well were sub-contracted out to the local firm Maghreb Projets A.C. Observation results are indicated in Figure 4.3-1.

The first half of observation works were carried out in the low water (dry) season, and indicate only the low level range of potentially available discharge. The latter half of observation works were done during the rainy season and the time of snow melt enabling overall a rough identification of the water level ~ discharge relationship over a somewhat wider range throughout the year. In the case of all gauging stations, discharge exhibited decrease during July~October, increase during November~April, and again decrease from May. Also, water level ~ discharge curve (H~Q) showed differing tendencies during the low water and high water seasons.

(2) Water Level Observations

Daily water level observations were commenced immediately after completion of gauging station installation (August 1996), and were carried out twice daily (07:00 and 17:00) for a one year period in the course of the Study. The purpose of the water level observations was to derive daily discharge from daily water level applying the water level ~ discharge curve. Continuous water level observations for a one year period enable the calculation of the daily discharge for the same one year period.

Daily water level observations were sub-contracted out by CDER to local villagers. Results of this observational data are expressed in diagram form in Figure 4.3-2~4.3-4 for daily average water level. In the case of the gauging stations at Infag and Arg, two peaks are seen, the first peak assumed to be immediate runoff as a result of rainfall during the rainy season, and the second peak being runoff form snow melt. In the case of Tidsi, this tendency was not significantly present.

(3) Water Lever ~ Discharge Curve

Results of discharge observation showed some spread in values; and this regard some minor correction in values was applied to results during the high water season to yield the water level ~ discharge curve indicated in Figure 4.3-5. Here, the H~Q curve is designated as secondary, with intercept of 0. The subsequent discharge ~ water level range of the curve established applying the size of observed discharge and limited number of observations (10 times) is narrow. This covers the important low water range under micro-hydropower generation planning; however, that for the high water range during times of flooding must be determined on the basis of extrapolation.

(4) Annual Discharge at New Gauging Stations

In the case of each gauging station, daily water level observation data calculated from the water level ~ discharge curve at each station in indicated in terms of the daily discharges set out in Figure 4.3-6~4.3-8. Although observation results for June and July have not as yet been recovered, factoring in of this data can be expected to result in a change in the configuration of the discharge curve and the values for design discharges Q_1~Q_{365}. Since this not as yet collected data corresponds to the latter half of the dry season, values for Q_{275} and after would become smaller. As indicated in the tables below, data as of May indicates a trend in larger design discharge at the new gauging stations than that estimated under the Prefeasibility Study for Adardour, Arg and Tidsi.

Amizmiz river (m³/s)

	Site	Generating discharge	Q_{185}	Q_{275}	Q_{329}
Pre F/S	Adardour	Q_{257} = 0.11	0.156	0.100	0.044
New gauging station	Infag		0.664	0.306	0.209
Site conversion	Adardour		0.193	0.089	0.061

Reraya river (m³/s)

	Site	Generating discharge	Q_{185}	Q_{275}	Q_{329}
Pre F/S	Arg	Q_{283} = 0.18	0.346	0.222	0.097
New gauging station	Arg		2.045	1.478	0.835

Zat river (m³/s)

	Site	Generating discharge	Q_{185}	Q_{275}	Q_{329}
Pre F/S	Tidsi	$Q_{198.5}$ = 0.15	0.165	0.083	0.044
New gauging station	Tidsi		0.350	0.261	0.141

As can be seen from the above, discharge estimated on the basis of existing data and that based on actual observations shows a considerable difference. As of this point in time it is difficult to judge which is the more accurate; however, to be on the safe side the data from existing gauging stations has been applied to planning under the Study.

4.4 Design Peak Flood Discharge and Flood Water Level

In order to protect intake and power generating facilities from flood damage, peak flood discharge and flood water level were estimated for intake and power house sites.

(1) Design Daily Rainfall

In order to estimate peak flood discharge, design daily rainfall was calculated on the basis of observation records available at the existing gauging stations to the downstream of the site catchment areas. Due to lacking observation data, the period used to determine design daily rainfall differs for each basin.

Figure 4.4-1 indicates daily rainfall with a return period of 1/2~1/100 years. In the case of return periods of 1/10 and 1/50, design daily rainfall fluctuates from 49.96~69.08 mm to 68.34~91.24 mm, respectively.

(2) Design Peak Flood Discharge

The peak flood discharge was estimated by rational formula. The said rational formula is relatively commonly applied in the case of small catchments in mountainous area. This approach represents a comparatively simple method of converting rainfall into runoff on the basis of estimation of flood concentration time and rainfall intensity during the flood concentration time period.

The Bayern and Kraven formula was applied to estimation of flood concentration time, with average rainfall intensity during the flood concentration time period being determined according to the Monobe formula. Runoff coefficient is 0.9, applicable to the hill and valley terrain of the subject catchments.

Table 4.4-1 indicates peak flood discharge in the case of return periods of 1/10, 1/50 and 1/100 for the intake and power house sites for the Adardour, Arg and Tidsi schemes. In the case of each return period, values show a tendency to be higher than the estimated peak values on the basis of momentary discharges observed at the existing gauging stations at the downstream of the same catchment.

(3) Design Peak Flood Water Level

Flood water depth for each design peak discharge for the return periods estimated in (2) above was calculated on the basis of the water level ~ discharge curve. In the case of Adardour and Arg, the river cross-section at the planned intake site is less than that at the power house site. As a result, water depth increase is greater at the power house site during times of flooding. In the case of Tidsi, a reverse situation is seen. Under the Adardour and Arg schemes, the differential between the intake site and power house site in water depth during flooding is small, with that for the former being 0.18~0.23 m, and that for the latter being 0.05~0.08 m. In the case of Tidsi, the difference is somewhat larger at 0.29~0.38.

Calculated flood water levels and flood concentration times were cross-checked through interview with village leaders and examination of flood traces in-situ.

Table 4.4-2 indicates flood discharge, water depth and water level at intake and power house sites for each return period.

4.5 River Water for Irrigation and Power Generation

In most of the villages, cropping pattern is similar (see Figure 2.1-3, Interim Report): Cereal crops (wheat & barley) are cultivated in November/December and harvested in May/June. Maize is grown in May - September. Most of the vegetables and nuts in May - August/September. Green vegetables in March - September. Potatoes in February - July. Most of the field lie fallow in September and October. The acreage under wheat/barley cultivation is large, about 77% of total cropped area. Tree crops and vegetables take up 16% and 7 % respectively. The tilth becomes small after September.

Irrigation is not required for wheat/barley and tree crops. A two-week irrigation rotation is commonly practiced for maize, potato and onion. Given a water supply of $0.1m^3/s$ and a 7-day rotation under an evapotranspiration regime of 8mm/day, the irrigable area is about 15ha per day, which is greater than the area under irrigation in May to September in most of the villages.

The active beneficiaries have recognized the importance to rotate water between irrigation and power generation. In cases when water for irrigation becomes scarce, with proper management and pre-determined rules to tide over the scarcity, water will not be used for power generation, except for a few hours in the twilight/evening zone or the pre-determined day of the week for power generation.

Apprehension on the impact on irrigation is unwarranted since in most cases discharge in river falls below $0.1m^3/s$ only very late in the dry season, in September to November, during which irrigation is not needed and area under irrigation in very limited.

Table 4.1-2 Monthly and Annual Mean Value – Microhydro Stations

unit=m3/s

Adardour Catchment=23km2 From Tahanaout / factor = 0.14

Year	1983	1984	1985	1986	1988	1989	1990	1991	1992	1993	Mean
Sept	0.02	0.02	0.02	0.02	0.06	0.11	0.15	0.05	0.04	0.01	0.05
Oct	0.01	0.01	0.10	0.05	0.19	0.80	0.05	0.08	0.03	0.01	0.14
Nov	0.06	0.03	0.08	0.12	0.67	0.32	0.05	0.02	0.01	0.17	0.16
Dec	0.06	0.01	0.09	0.07	0.29	0.41	0.07	0.62	0.05	0.13	0.18
Jan	0.06	0.33	0.10	0.10	0.18	0.48	0.05	0.03	0.06	0.13	0.15
Feb	0.01	0.21	0.10	0.73	0.18	0.38	0.13	0.04	0.06	0.25	0.21
Mar	0.07	0.15	0.16	0.26	0.34	0.79	0.81	0.14	0.18	0.85	0.38
Apr	0.13	0.52	0.15	0.34	0.82	0.36	0.99	1.83	0.30	0.92	0.63
May	0.44	1.01	0.31	0.33	0.88	0.65	0.48	2.13	0.32	0.76	0.73
Jun	0.10	0.35	0.14	0.11	0.67	0.34	0.26	1.02	0.24	0.16	0.34
Jul	0.04	0.06	0.04	0.05	0.29	0.10	0.13	0.11	0.14	0.03	0.10
Aug	0.01	0.03	0.01	0.02	0.13	0.05	0.06	0.11	0.03	0.05	0.05
Annual	0.09	0.23	0.11	0.18	0.39	0.40	0.27	0.52	0.13	0.29	0.26

Inzaine Catchment=79km2 From Tahanaout / factor = 0.47

Year	1983	1984	1985	1986	1988	1989	1990	1991	1992	1993	Mean
Sept	0.07	0.08	0.05	0.05	0.19	0.36	0.51	0.17	0.13	0.05	0.17
Oct	0.15	0.02	0.33	0.17	0.65	2.69	0.18	0.26	0.10	0.03	0.46
Nov	0.19	0.10	0.25	0.42	2.25	1.08	0.17	0.07	0.12	0.57	0.52
Dec	0.20	0.14	0.30	0.24	0.97	1.38	0.23	2.07	0.15	0.42	0.61
Jan	0.20	1.10	0.32	0.33	0.67	1.62	0.18	0.11	0.21	0.43	0.51
Feb	0.14	0.70	0.35	2.45	0.60	1.28	0.43	0.14	0.21	0.85	0.69
Mar	0.24	0.52	0.55	0.88	1.13	2.66	2.83	0.48	0.62	2.84	1.28
Apr	0.42	1.74	0.49	1.13	2.75	1.22	3.33	6.15	1.01	3.07	2.13
May	1.49	3.49	1.01	1.11	2.96	2.19	1.61	7.14	1.09	2.56	2.47
Jun	0.33	1.18	0.46	0.37	2.24	1.13	0.86	3.41	0.82	0.54	1.13
Jul	0.12	0.20	0.15	0.16	0.97	0.34	0.43	0.46	0.47	0.10	0.34
Aug	0.04	0.09	0.05	0.06	0.43	0.15	0.21	0.36	0.10	0.18	0.17
Annual	0.30	0.78	0.36	0.60	1.31	1.35	0.91	1.74	0.42	0.97	0.87

Arg Catchment=45km2 From Tahanaout / factor = 0.31

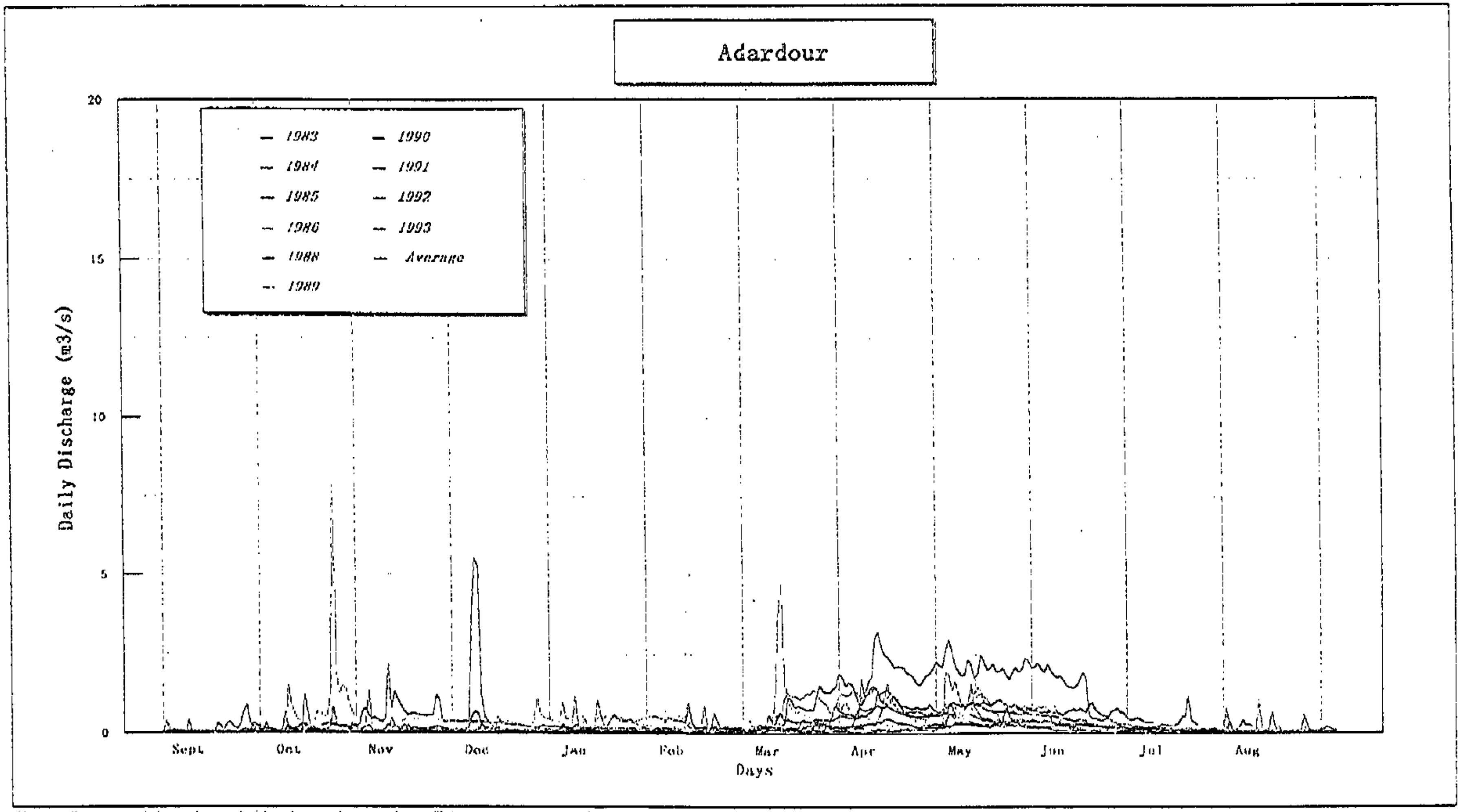
Year	1983	1984	1985	1986	1988	1989	1990	1991	1992	1993	Mean
Sept	0.05	0.05	0.04	0.03	0.15	0.24	0.33	0.11	0.09	0.03	0.11
Oct	0.10	0.01	0.22	0.11	0.43	1.77	0.12	0.17	0.07	0.02	0.30
Nov	0.13	0.06	0.17	0.28	1.49	0.71	0.11	0.05	0.08	0.37	0.34
Dec	0.13	0.09	0.20	0.16	0.64	0.91	0.15	1.37	0.10	0.28	0.40
Jan	0.13	0.72	0.21	0.22	0.41	1.07	0.12	0.07	0.14	0.28	0.34
Feb	0.09	0.46	0.23	1.62	0.40	0.84	0.28	0.09	0.14	0.56	0.46
Mar	0.16	0.34	0.36	0.58	0.75	1.76	1.87	0.32	0.41	1.87	0.84
Apr	0.28	1.15	0.32	0.74	1.82	0.80	2.19	4.06	0.67	2.03	1.41
May	0.98	2.30	0.69	0.73	1.95	1.44	1.06	4.71	0.72	1.69	1.63
Jun	0.22	0.78	0.30	0.25	1.48	0.75	0.57	2.25	0.54	0.35	0.75
Jul	0.08	0.13	0.10	0.11	0.64	0.22	0.28	0.31	0.31	0.07	0.23
Aug	0.03	0.06	0.03	0.04	0.28	0.10	0.14	0.24	0.07	0.12	0.11
Annual	0.20	0.51	0.24	0.40	0.87	0.89	0.60	1.15	0.28	0.64	0.58

Anfli Catchment=134km2 From Aghbalou / factor = 0.34

Year	1981	1982	1983	1984	1988	1990	1991	1992	1993	1994	Mean
Sept	0.03	0.13	0.02	0.06	0.25	3.30	0.72	0.28	0.06	0.12	0.50
Oct	0.28	0.14	0.26	0.04	0.75	0.73	0.91	0.37	0.41	0.28	0.42
Nov	0.45	0.17	0.86	0.34	2.76	0.23	0.37	0.31	1.91	0.36	0.78
Dec	0.25	0.18	0.26	0.60	0.91	0.82	12.76	0.46	1.61	0.23	1.81
Jan	0.41	0.14	0.17	1.99	0.86	0.25	1.72	0.53	1.91	0.13	0.81
Feb	0.72	0.24	0.13	2.07	1.00	4.06	1.45	0.62	3.90	0.52	1.46
Mar	1.12	0.34	0.38	1.73	1.67	25.63	10.10	3.49	11.12	1.18	5.68
Apr	3.96	0.40	0.91	2.90	2.56	20.69	24.10	3.36	8.67	5.44	7.30
May	5.30	0.57	1.80	4.28	1.91	4.53	21.07	1.32	5.84	2.85	4.95
Jun	1.00	0.09	0.29	1.13	1.08	0.69	4.16	0.27	1.27	0.56	1.05
Jul	0.27	0.03	0.04	0.40	4.78	0.55	0.75	0.09	0.30	0.30	0.75
Aug	0.19	0.01	0.01	0.13	0.33	0.86	1.76	0.07	0.20	2.18	0.57
Annual	1.17	0.20	0.43	1.30	1.58	5.19	6.68	0.93	3.09	1.18	2.17

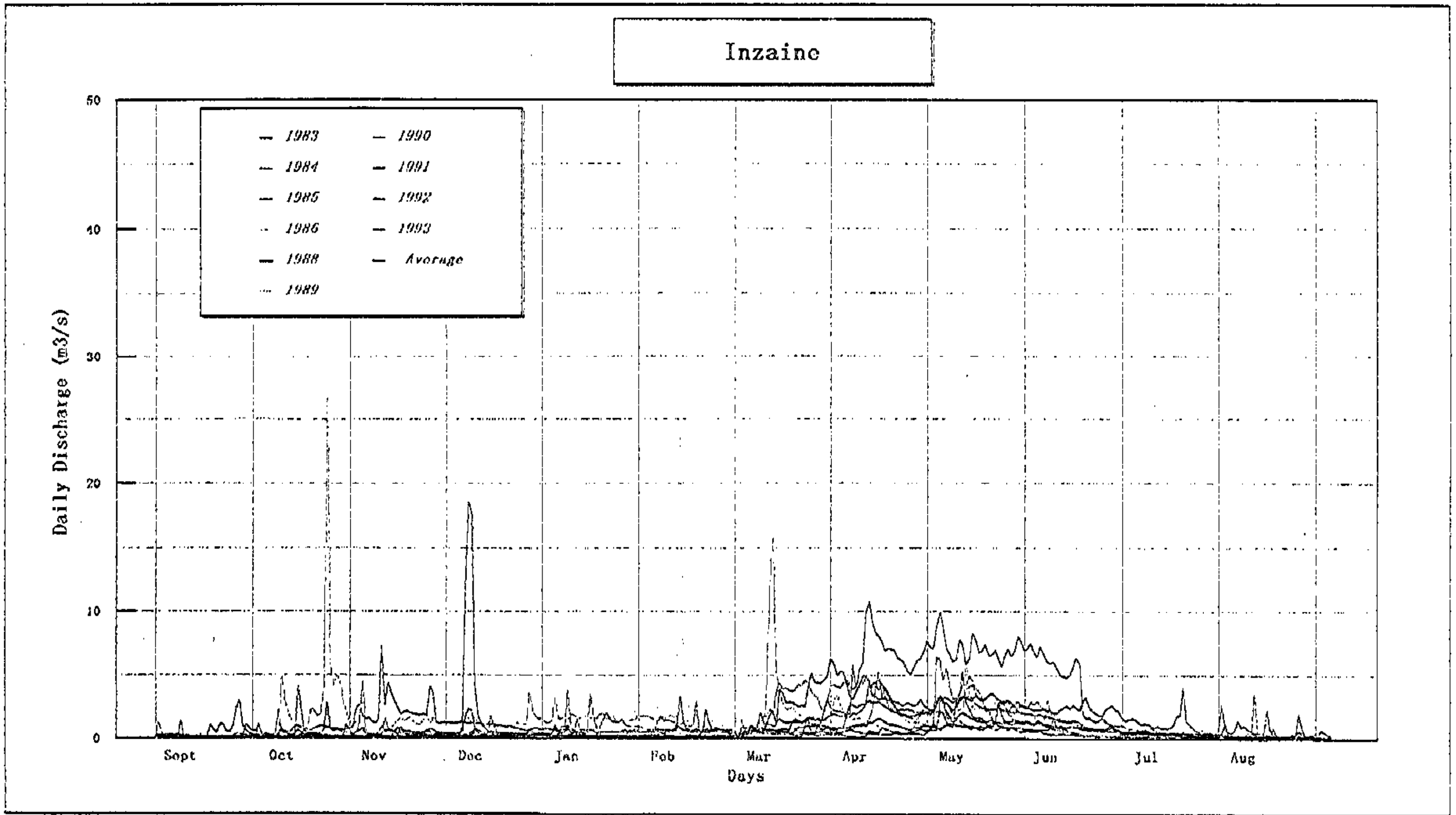
Tidsi Catchment=24km2 From Aghbalou / factor = 0.06

Year	1981	1982	1983	1984	1988	1990	1991	1992	1993	1994	Average
Sept	0.01	0.02	0.00	0.01	0.04	0.58	0.13	0.05	0.01	0.02	0.09
Oct	0.05	0.02	0.05	0.01	0.13	0.13	0.16	0.07	0.07	0.05	0.07
Nov	0.08	0.03	0.15	0.06	0.49	0.04	0.06	0.05	0.34	0.06	0.14
Dec	0.04	0.03	0.05	0.11	0.16	0.15	2.25	0.08	0.28	0.04	0.32
Jan	0.07	0.02	0.03	0.35	0.15	0.04	0.30	0.09	0.34	0.02	0.14
Feb	0.13	0.04	0.02	0.37	0.18	0.72	0.26	0.11	0.69	0.09	0.26
Mar	0.20	0.06	0.07	0.31	0.29	4.52	1.78	0.62	1.96	0.21	1.00
Apr	0.70	0.07	0.17	0.51	0.45	3.65	4.25	0.59	1.53	0.96	1.29
May	0.94	0.10	0.32	0.75	0.34	0.80	3.72	0.23	1.03	0.50	0.87
Jun	0.18	0.02	0.05	0.20	0.19	0.12	0.73	0.05	0.22	0.10	0.19
Jul	0.05	0.00	0.01	0.07	0.84	0.10	0.13	0.02	0.05	0.05	0.13
Aug	0.03	0.00	0.00	0.02	0.06	0.15	0.31	0.01	0.04	0.38	0.10
Annual	0.21	0.04	0.08	0.23	0.28	0.92	1.18	0.16	0.55	0.21	0.38



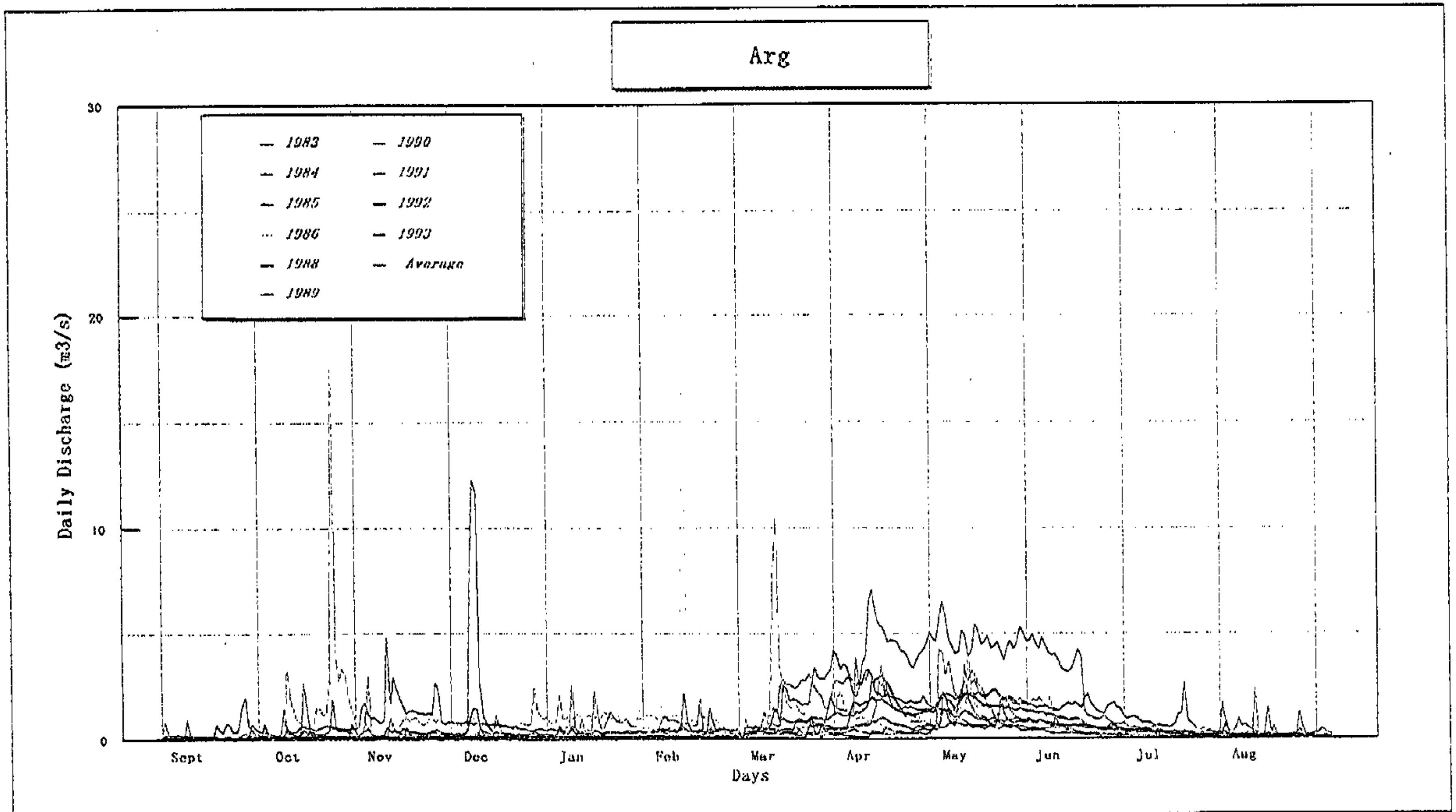
Note: Generated based on daily data observed at Tahanout using a Factor= 0.14

Figure 4.1-1 Daily Discharge – Adardour



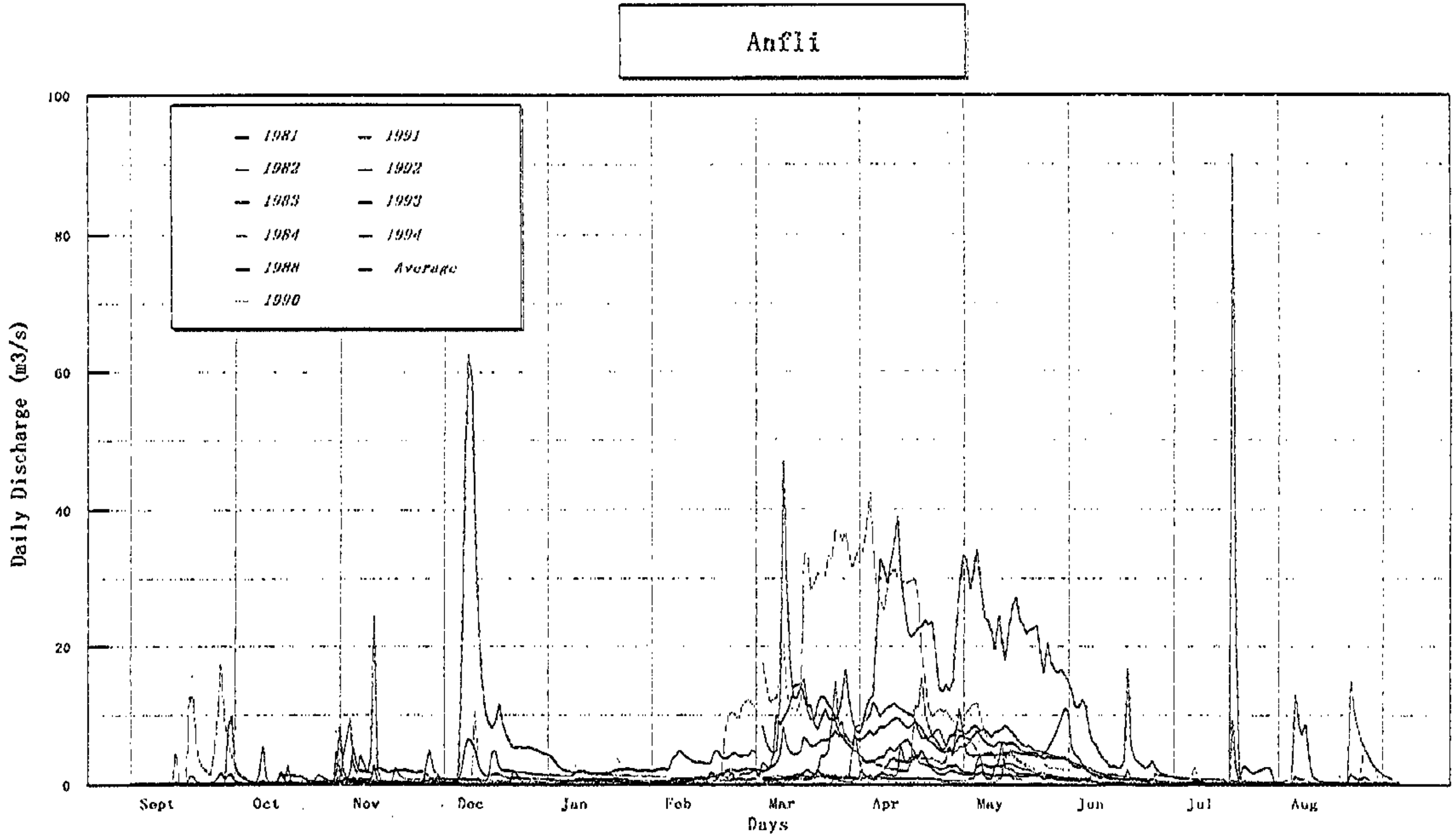
Note: Generated based on daily data observed at Tahanout using a factor= 0.47

Figure 4.1-2 Daily Discharge – Inzaine



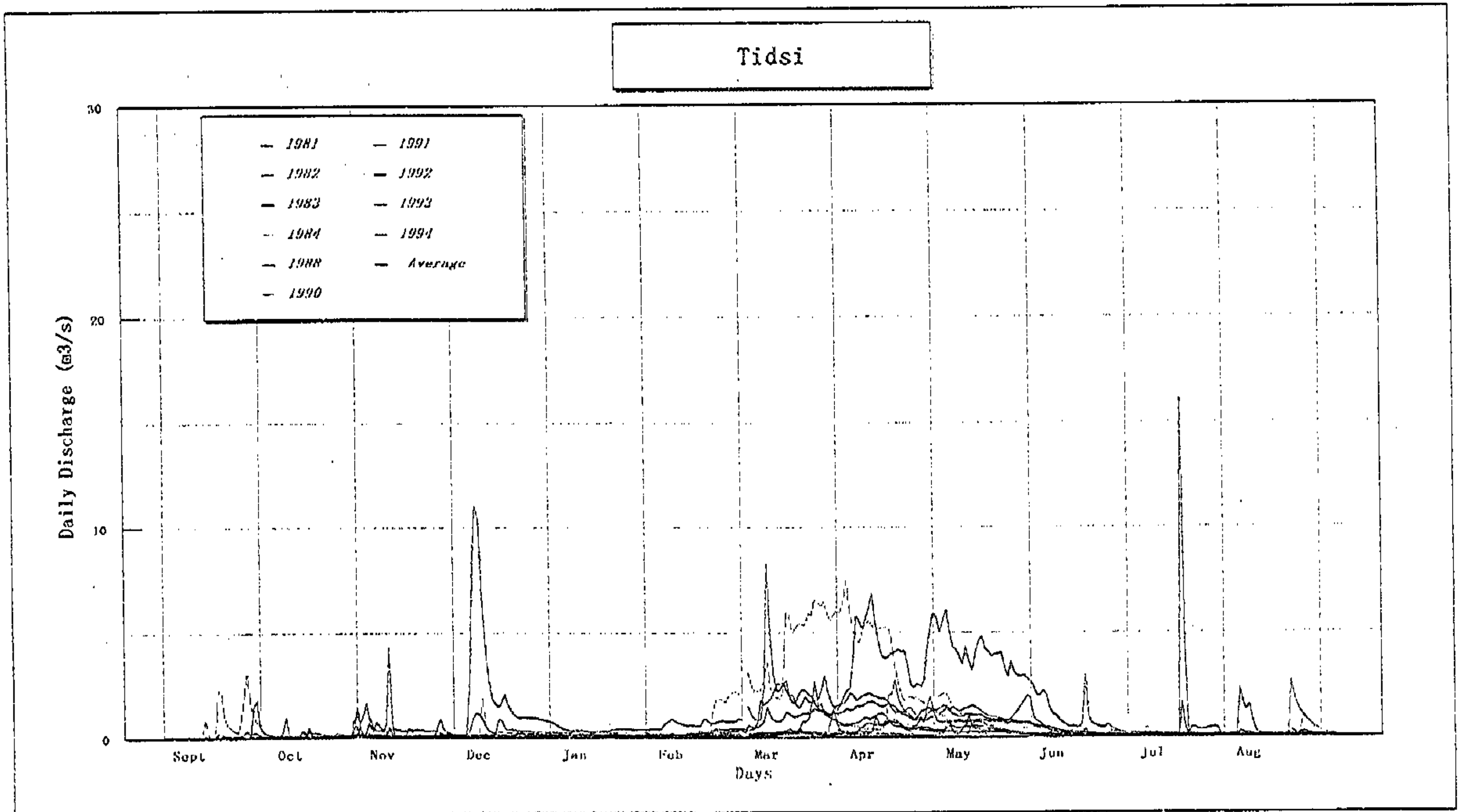
Note: Generated based on daily data observed at Tahanout using a factor= 0.31

Figure 4.1-3 Daily Discharge – Arg



Note: Generated based on daily data observed at Aghbalou using a factor= 0.34

Figure 4.1-4 Daily Discharge – Anfli



Note: Generated based on daily data observed at Aghbalou using a factor= 0.06

Figure 4.1-5 Daily Discharge – Tidsi

Table 4.1-3 Design Discharge – Mycrohydro Stations

Unit=m3/s

Adardour Catchment=23km2 From Tahanaout / factor = 0.14

Year	1983	1984	1985	1986	1988	1989	1990	1991	1992	1993	Mean
Q1	1.582	1.918	1.229	8.358	2.184	7.952	1.890	5.544	0.455	1.470	0.987
Q35	0.241	0.805	0.221	0.426	0.885	0.700	0.795	2.044	0.309	1.093	0.654
Q95	0.079	0.258	0.134	0.237	0.655	0.469	0.321	0.304	0.206	0.384	0.397
Q185	0.051	0.091	0.084	0.062	0.253	0.339	0.074	0.081	0.060	0.104	0.156
Q275	0.022	0.025	0.041	0.044	0.158	0.138	0.052	0.037	0.038	0.021	0.100
Q329	0.011	0.006	0.013	0.010	0.068	0.054	0.042	0.017	0.031	0.011	0.044
Q355	0.006	0.005	0.008	0.007	0.038	0.039	0.024	0.011	0.027	0.009	0.031
Q365	0.004	0.003	0.002	0.006	0.018	0.029	0.016	0.008	0.022	0.007	0.027

Inzaine Catchment=79km2 From Tahanaout / factor = 0.470

Year	1983	1984	1985	1986	1988	1989	1990	1991	1992	1993	Mean
Q1	5.311	6.439	4.127	28.059	7.332	26.696	6.345	18.612	1.528	4.935	3.315
Q35	0.808	2.703	0.743	1.429	2.970	2.350	2.670	6.862	1.039	3.671	2.195
Q95	0.265	0.865	0.451	0.794	2.200	1.575	1.076	1.020	0.691	1.288	1.331
Q185	0.171	0.304	0.281	0.208	0.851	1.137	0.249	0.274	0.202	0.348	0.524
Q275	0.074	0.082	0.138	0.147	0.531	0.463	0.174	0.125	0.127	0.072	0.337
Q329	0.038	0.021	0.045	0.032	0.227	0.181	0.141	0.057	0.104	0.038	0.147
Q355	0.019	0.017	0.028	0.024	0.128	0.130	0.081	0.035	0.091	0.029	0.103
Q365	0.013	0.011	0.008	0.019	0.062	0.096	0.055	0.027	0.074	0.025	0.089

Arg Catchment=48km2 From Tahanaout / factor = 0.310

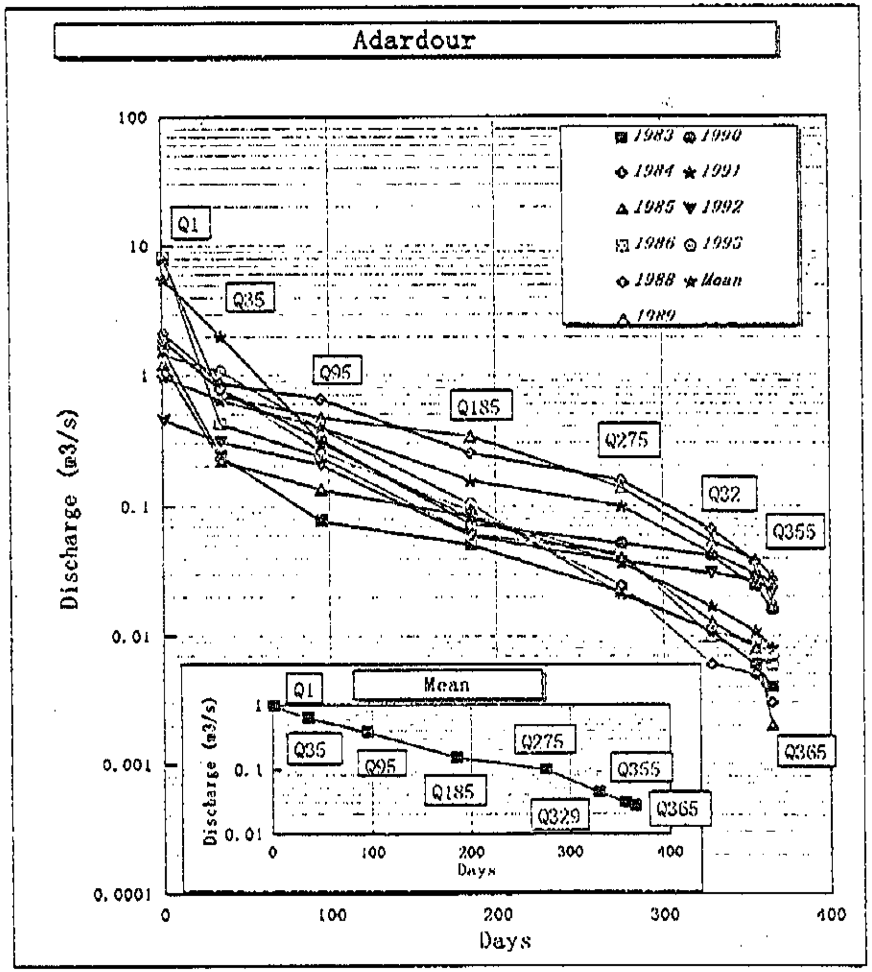
Year	1983	1984	1985	1986	1988	1989	1990	1991	1992	1993	Mean
Q1	3.503	4.247	2.722	18.507	4.836	17.608	4.185	12.276	1.008	3.255	2.186
Q35	0.533	1.783	0.490	0.942	1.959	1.550	1.761	4.526	0.685	2.421	1.448
Q95	0.175	0.570	0.298	0.524	1.451	1.039	0.710	0.673	0.456	0.849	0.878
Q185	0.113	0.201	0.185	0.137	0.561	0.750	0.164	0.180	0.133	0.229	0.346
Q275	0.049	0.054	0.091	0.097	0.350	0.305	0.115	0.082	0.084	0.047	0.222
Q329	0.025	0.014	0.030	0.021	0.150	0.119	0.093	0.038	0.069	0.025	0.097
Q355	0.012	0.011	0.019	0.016	0.084	0.086	0.054	0.023	0.060	0.019	0.068
Q365	0.009	0.007	0.005	0.013	0.041	0.063	0.036	0.018	0.049	0.016	0.059

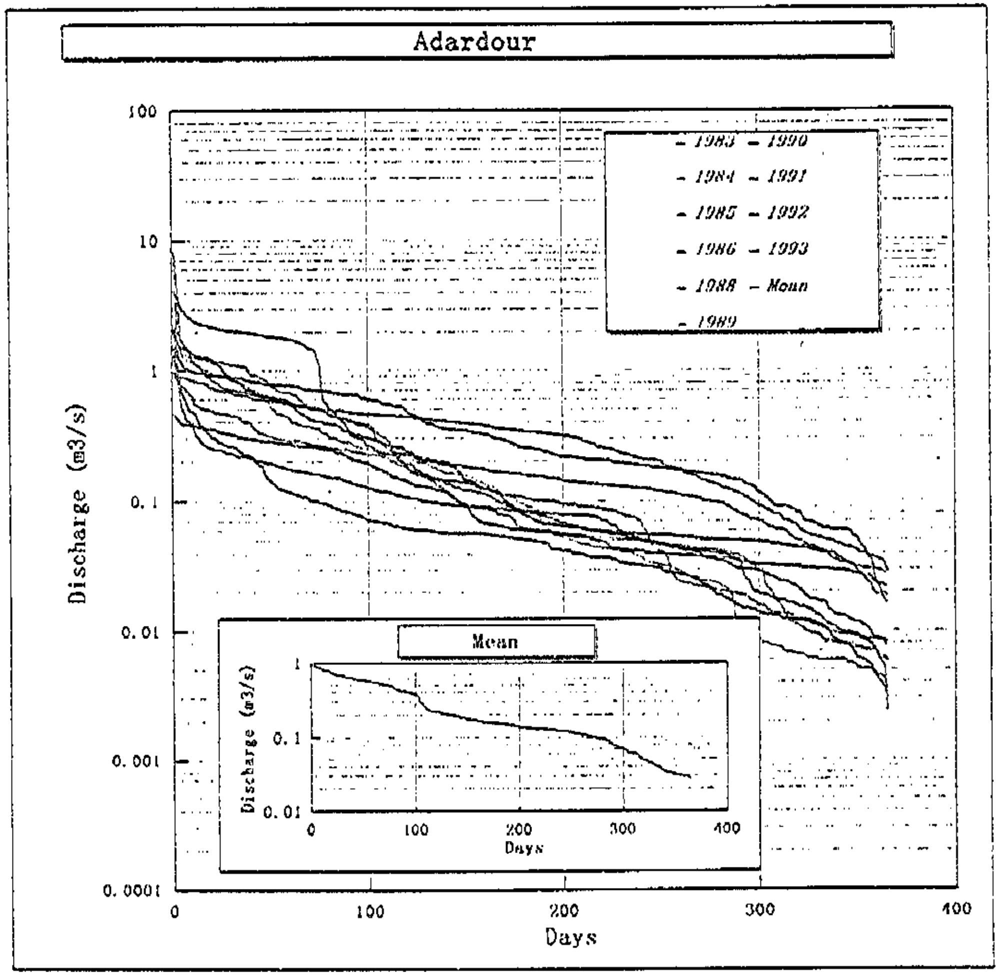
Anfli Catchment=134km2 From Aghbalou / factor = 0.340

Year	1981	1982	1983	1984	1988	1990	1991	1992	1993	1994	Mean
Q1	11.186	4.964	6.052	8.364	91.460	42.500	62.560	15.028	46.920	15.504	9.679
Q35	4.284	0.388	1.074	3.053	2.771	20.638	22.746	2.244	8.466	3.978	6.529
Q95	1.023	0.209	0.619	2.081	1.350	4.454	9.452	0.782	4.386	0.925	2.809
Q185	0.367	0.147	0.161	0.629	0.891	0.517	1.455	0.313	1.724	0.360	0.931
Q275	0.219	0.108	0.033	0.157	0.479	0.236	0.544	0.255	0.234	0.200	0.468
Q329	0.043	0.022	0.015	0.041	0.261	0.169	0.268	0.067	0.077	0.116	0.248
Q355	0.012	0.010	0.009	0.009	0.229	0.093	0.091	0.049	0.062	0.095	0.117
Q365	0.009	0.007	0.004	0.000	0.127	0.007	0.046	0.043	0.040	0.090	0.099

Tidsi Catchment=24km2 From Aghbalou / factor = 0.060

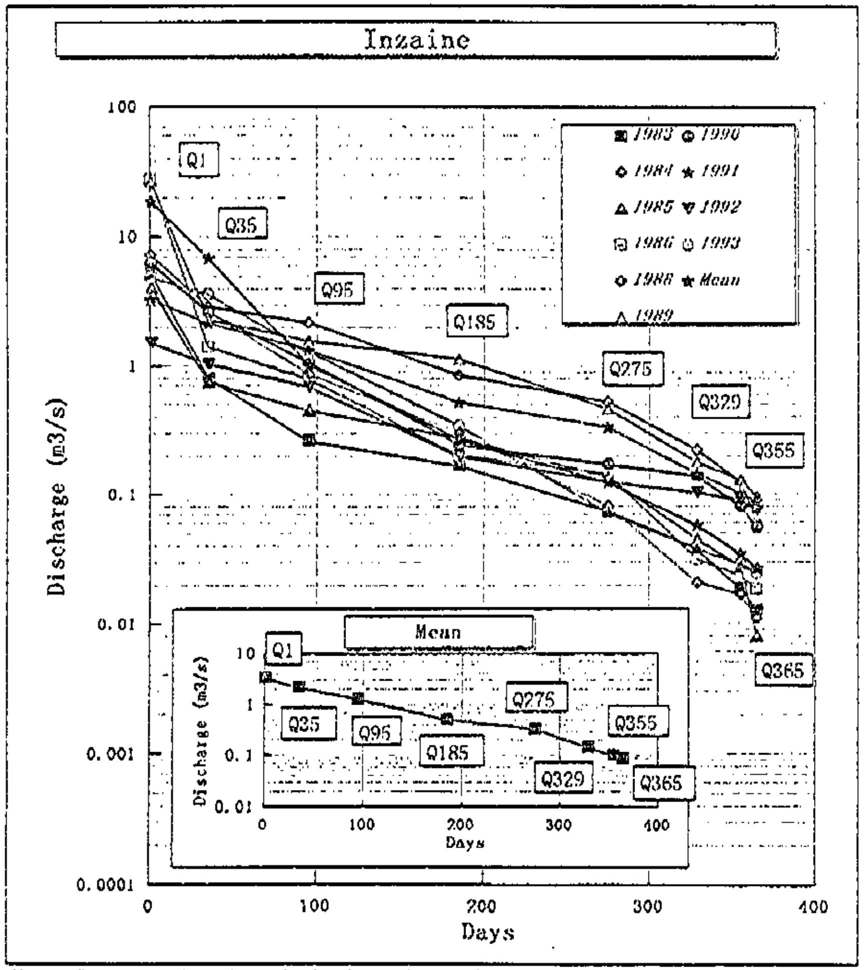
Year	1981	1982	1983	1984	1988	1990	1991	1992	1993	1994	Mean
Q1	1.974	0.876	1.068	1.476	16.140	7.500	11.040	2.652	8.280	2.736	1.708
Q35	0.756	0.068	0.190	0.539	0.489	3.642	4.014	0.396	1.494	0.702	1.152
Q95	0.181	0.037	0.109	0.367	0.238	0.786	1.668	0.138	0.774	0.163	0.496
Q185	0.065	0.026	0.028	0.111	0.157	0.091	0.257	0.055	0.304	0.064	0.165
Q275	0.039	0.019	0.006	0.028	0.085	0.042	0.096	0.045	0.041	0.035	0.083
Q329	0.008	0.004	0.003	0.007	0.047	0.030	0.047	0.012	0.014	0.021	0.044
Q355	0.002	0.002	0.002	0.002	0.040	0.016	0.016	0.009	0.011	0.017	0.021
Q365	0.002	0.001	0.001	0.000	0.022	0.001	0.008	0.008	0.007	0.016	0.017





Note: Generated based on daily data observed at Tahanaout using a factor= 0.14

Figure 4.1-6 Flow Duration Curve - Adardour



Note: Generated based on daily data observed at Tahanaout using a factor= 0.47

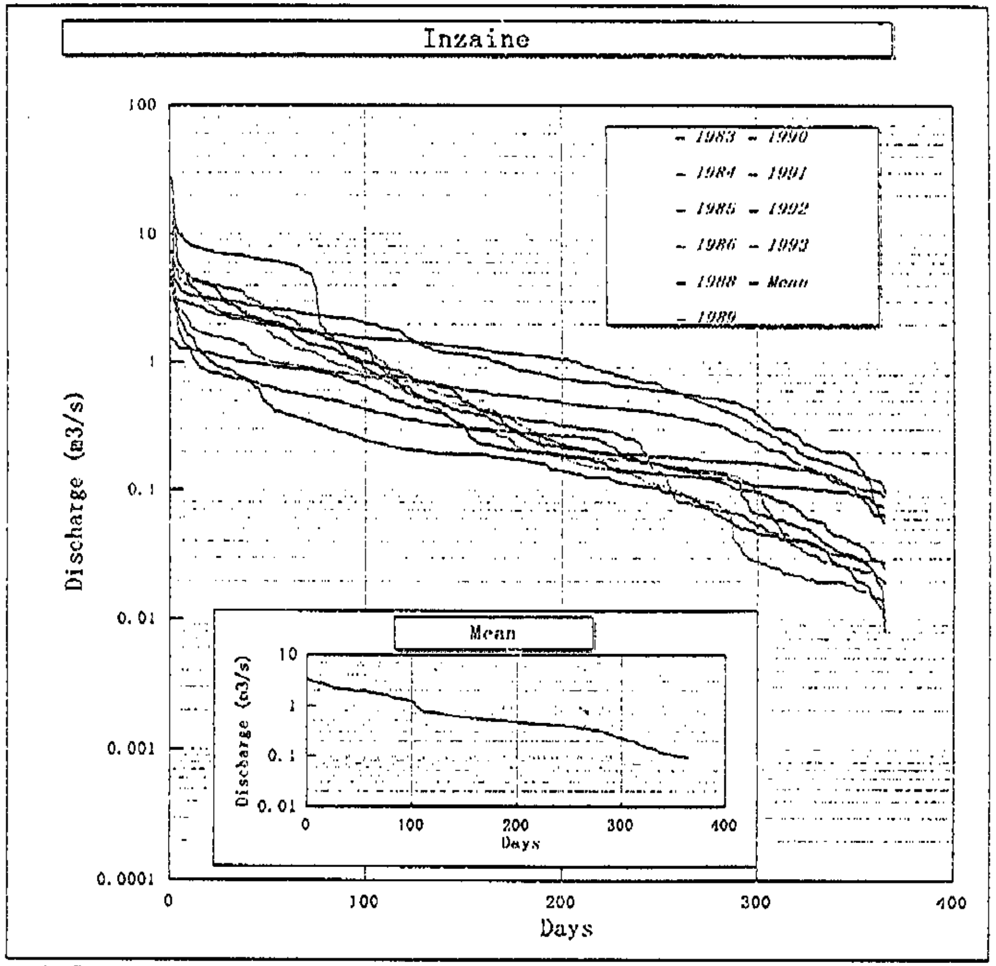
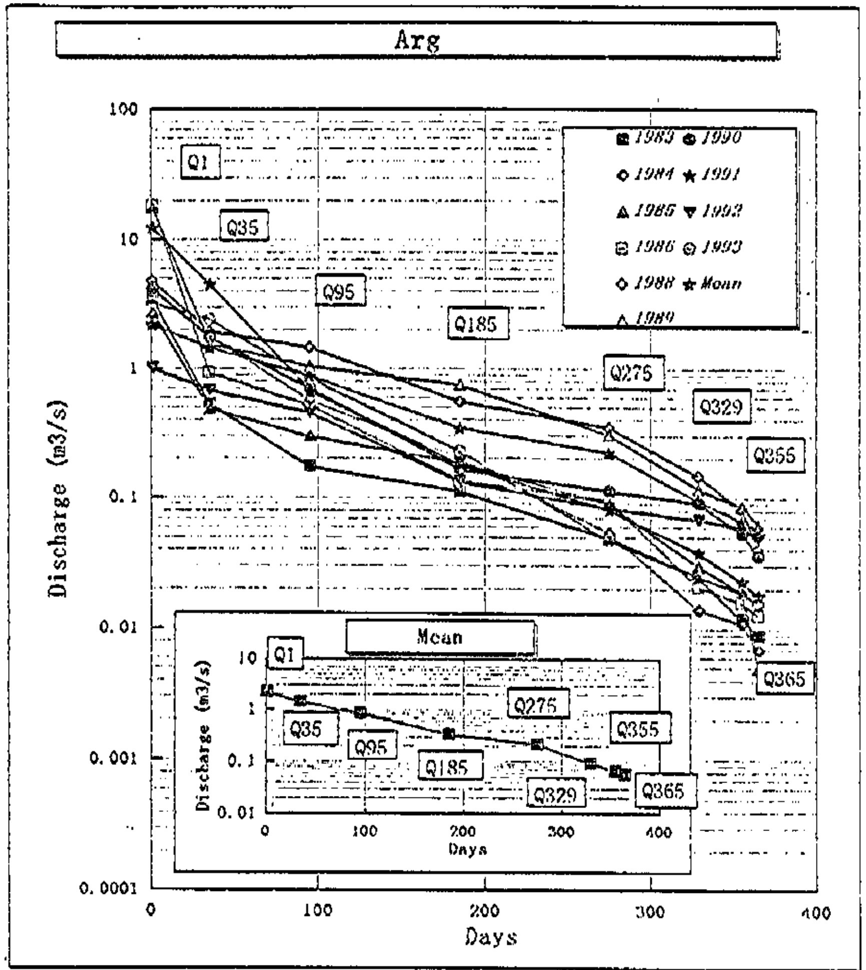
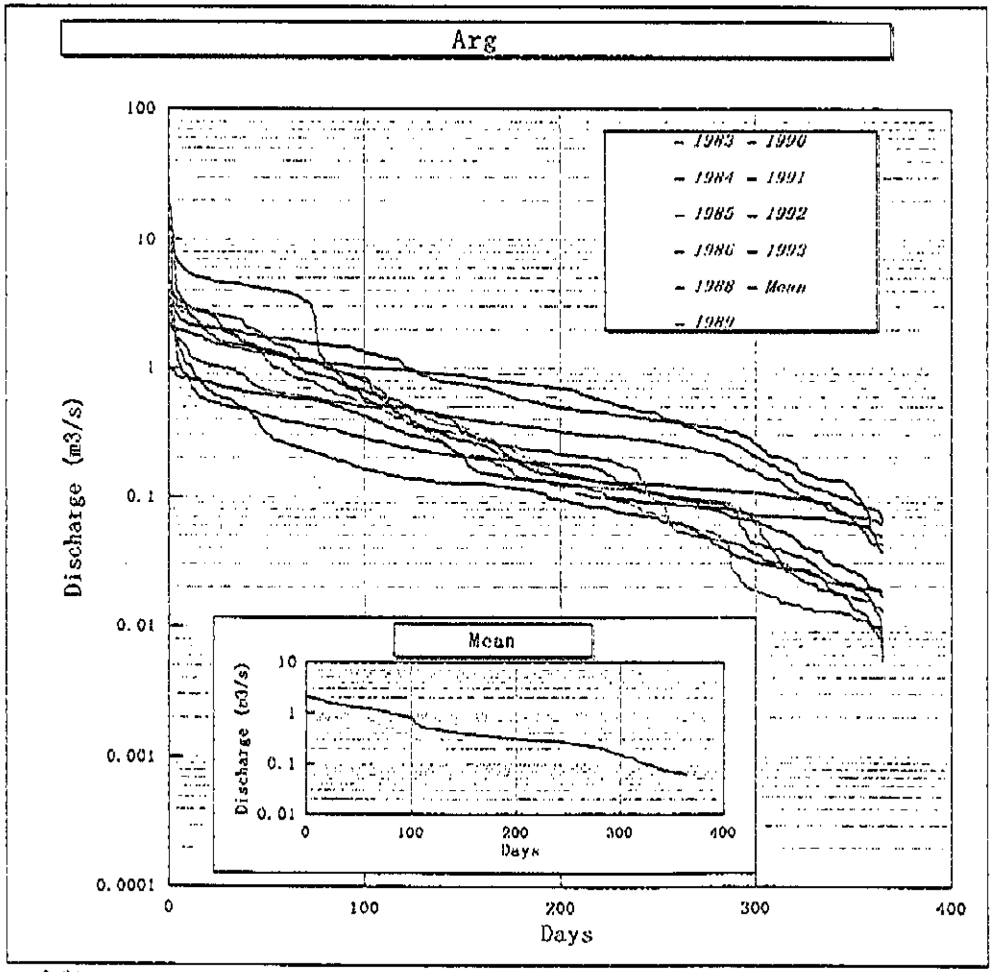


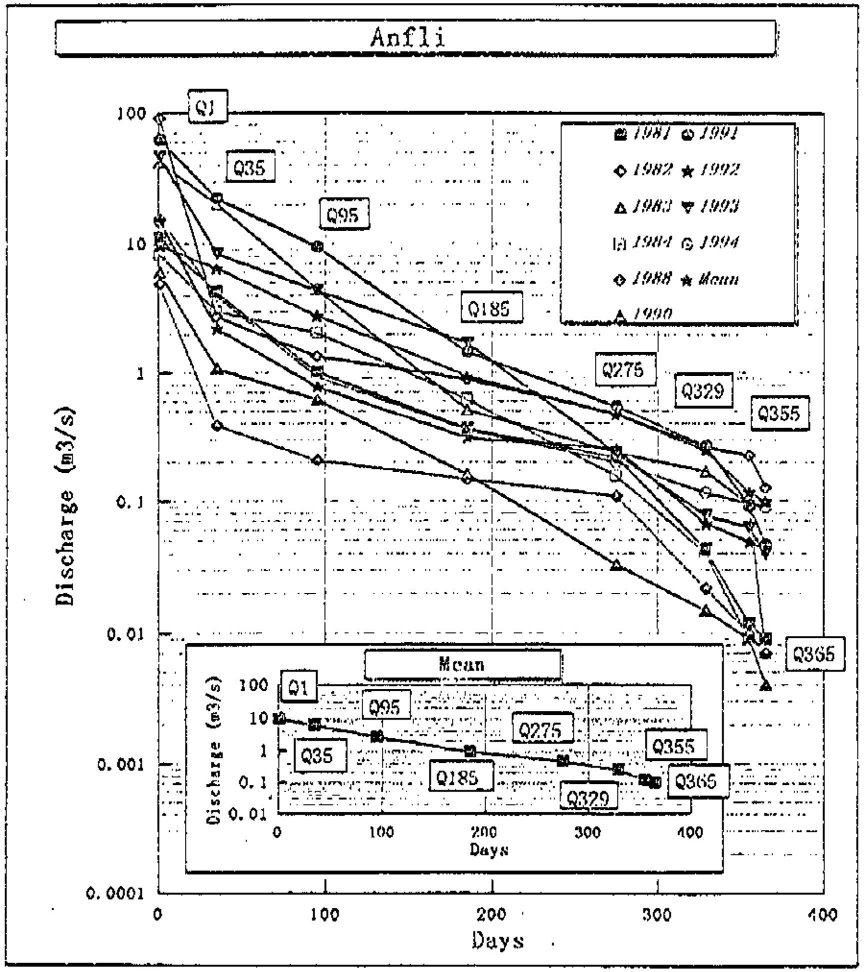
Figure 4.1-7 Flow Duration Curve – Inzaine

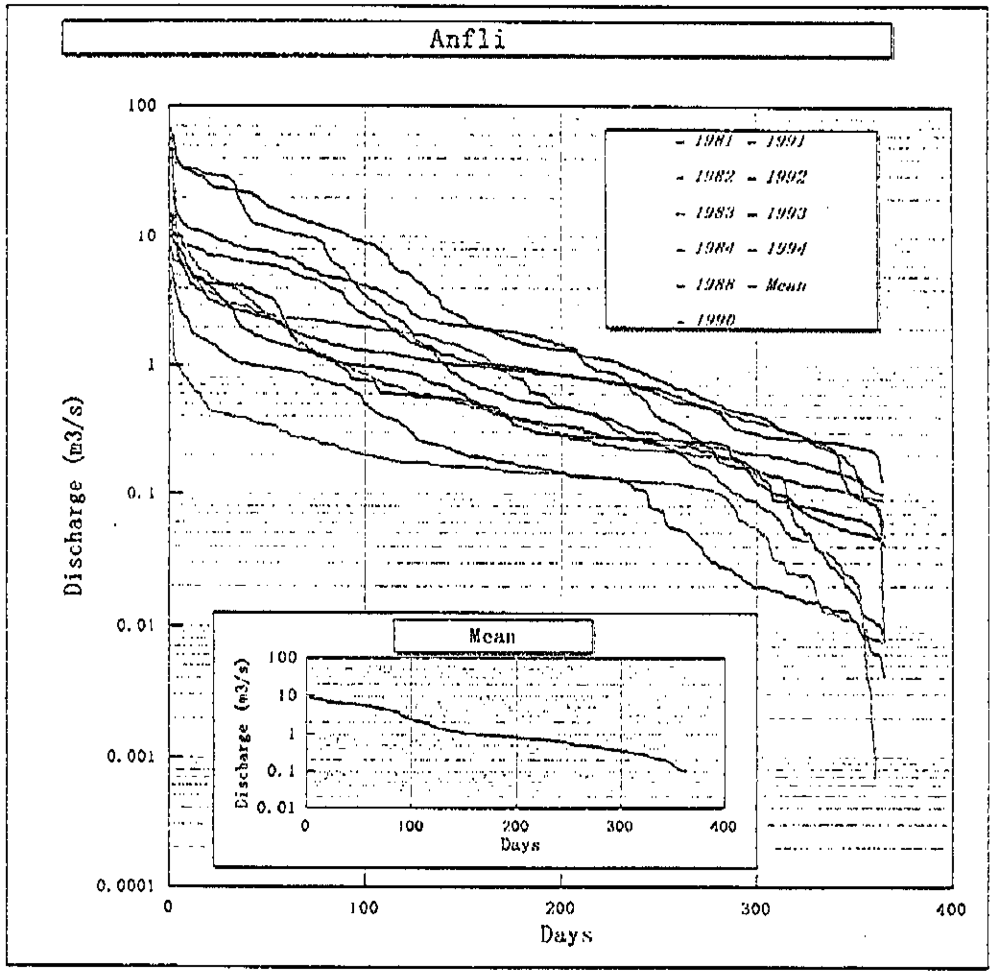




Note: Generated based on daily data observed at Tahanaout using a factor= 0.31

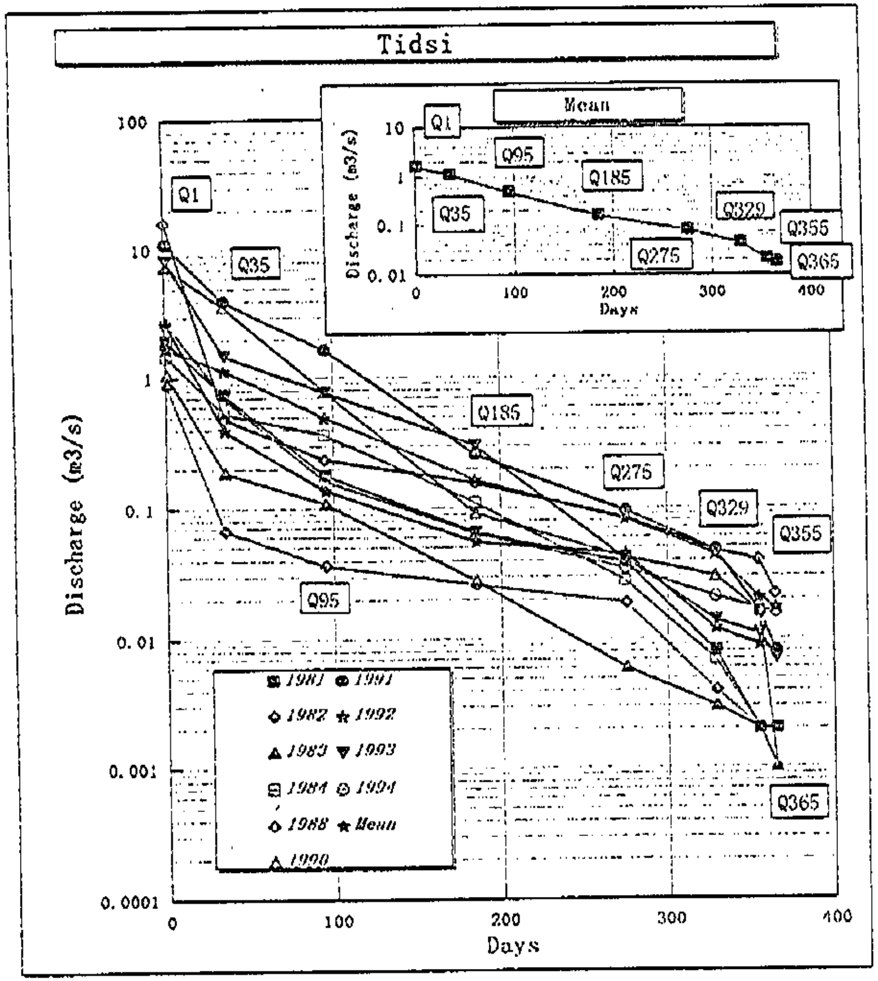
Figure 4.1-8 Flow Duration Curve – Arg

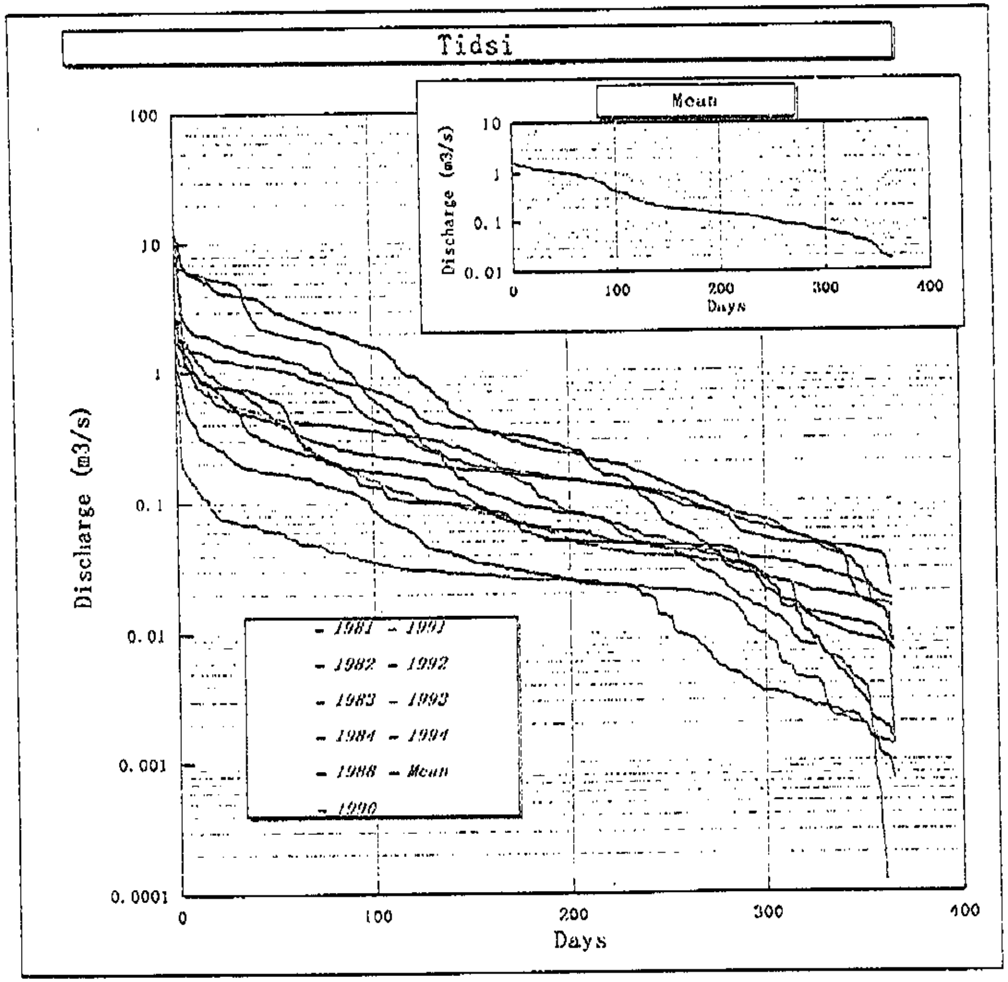




Note: Generated based on daily data observed at Aghbalou using a factor= 0.34

Figure 4.1-9 Flow Duration Curve – Anfli



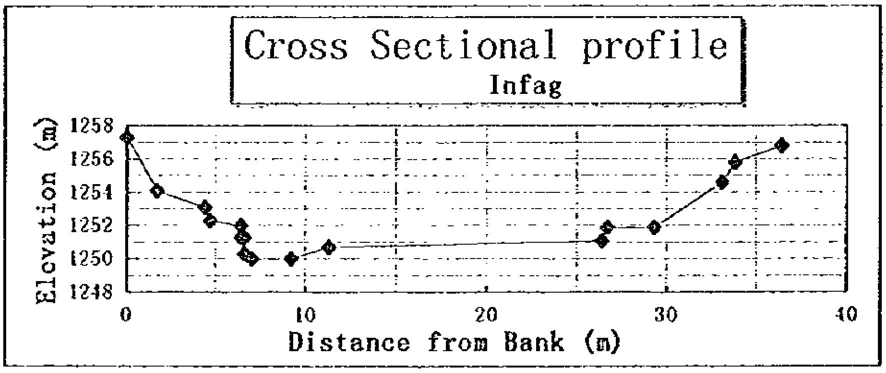


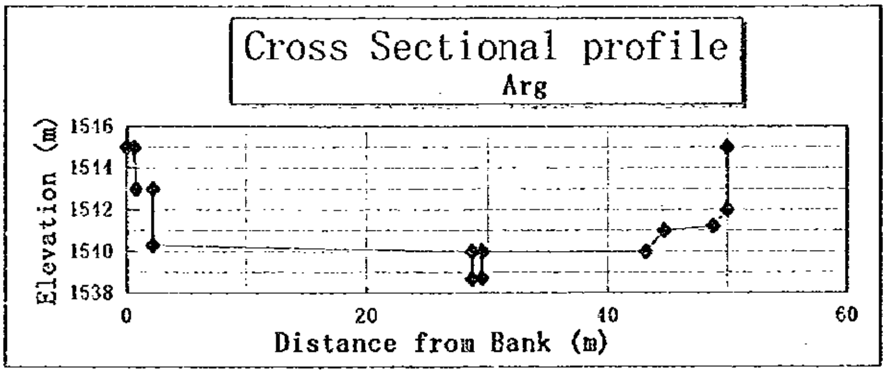
Note: Generated based on daily data observed at Aghbalou using a factor= 0.06

Figure 4.1-10 Flow Duration Curve – Tidsi

Cross Sectional Profile - Hydrometric Station

Station	Infag		Arg		Tidsi	
Elevation	1,250.0 m		1,538.7 m		1,722.0 m	
Coordinate	x	y	x	y	x	y
1	0.00	1,257.30	0.00	1,545.00	0.00	1,726.00
2	1.70	1,254.10	0.60	1,545.00	0.00	1,725.00
3	4.40	1,253.10	0.80	1,543.00	2.00	1,725.00
4	4.70	1,252.30	2.20	1,543.00	2.00	1,722.00
5	6.40	1,252.00	2.20	1,540.30	3.00	1,722.00
6	6.40	1,251.30	28.70	1,540.00	4.00	1,722.12
7	6.60	1,251.30	28.70	1,538.70	5.00	1,722.23
8	6.60	1,250.30	29.50	1,538.71	6.00	1,722.28
9	7.00	1,250.00	29.50	1,540.00	7.00	1,722.30
10	9.20	1,250.01	43.20	1,540.00	7.00	1,725.00
11	11.30	1,250.70	44.70	1,541.00	9.00	1,725.00
12	26.40	1,251.10	48.80	1,541.20	9.00	1,726.00
13	26.70	1,251.90	50.00	1,542.00		
14	29.30	1,251.90	50.00	1,545.00		
15	33.10	1,254.60				
16	33.80	1,255.80				
17	36.40	1,256.80				





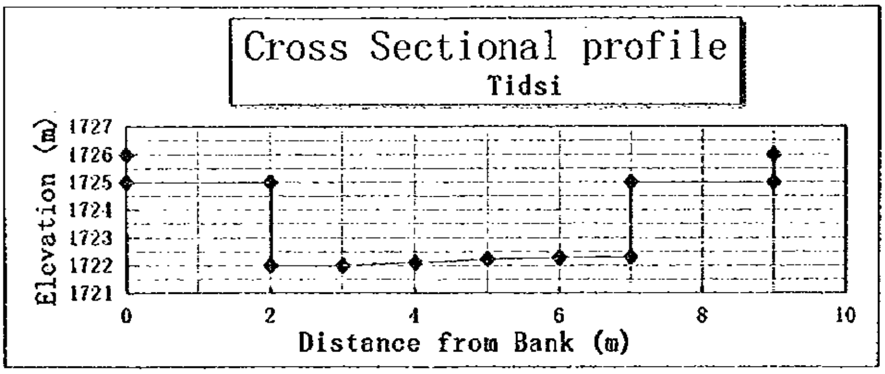
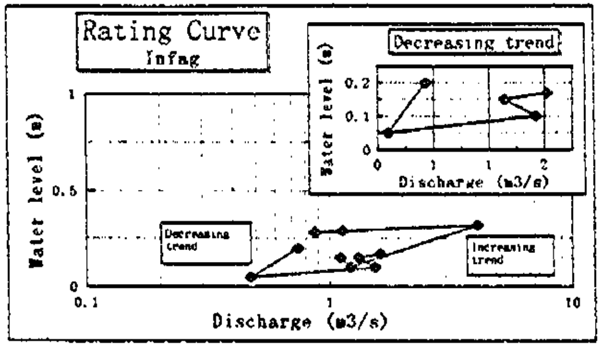
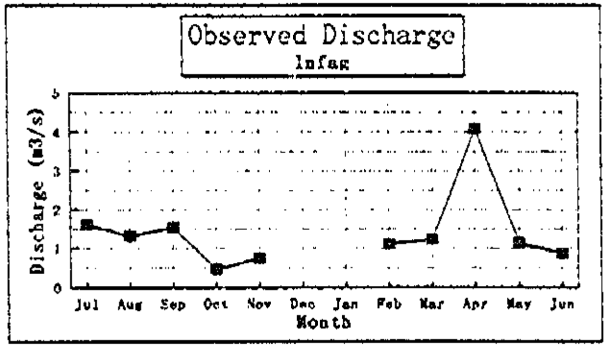


Figure 4.2-1 Cross Sectional Profile – Hydrometric Stations

Infag Anougal 79 km2

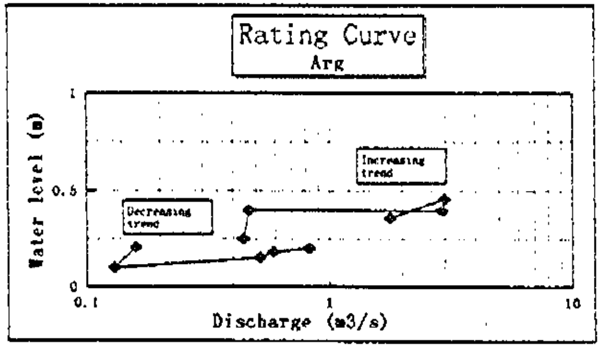
	Mean velocity (m/s)	Flow area (m2)	Discharge (m3/s)	Water Level (m)	Date
Jul	1.71	0.95	1.62	0.17	15/07/98
Aug	1.61	0.82	1.31	0.15	01/08/96
Sep	1.51	1.02	1.54	0.10	04/09/96
Oct	0.34	1.41	0.47	0.05	30/09/96
Nov	0.63	1.17	0.74	0.20	29/11/96
Dec					
Jan					
Feb	0.79	1.40	1.11	0.15	04/02/97
Mar	0.88	1.40	1.22	0.10	11/03/97
Apr	1.93	2.12	4.08	0.32	09/04/97
May	0.95	1.20	1.13	0.29	19/05/97
Jun	0.73	1.20	0.87	0.28	03/06/97

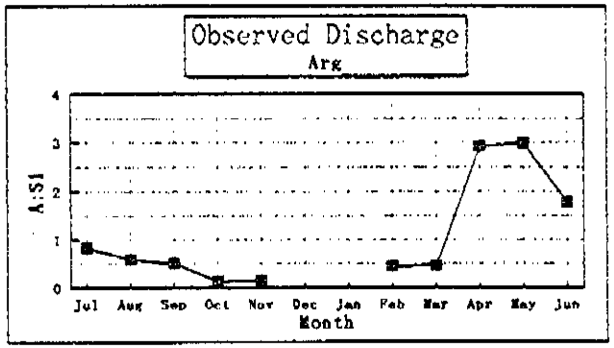




Arg Imenane 48 km2

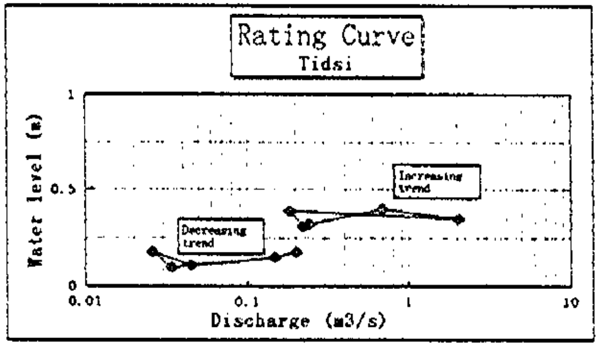
	Mean velocity (m/s)	Flow area (m2)	Discharge (m3/s)	Water depth (m)	Date
Jul	1.32	0.63	0.83	0.20	16/07/98
Aug	0.93	0.63	0.59	0.18	03/08/96
Sep	0.80	0.64	0.52	0.15	05/09/96
Oct	0.31	0.41	0.13	0.10	01/10/96
Nov	0.23	0.68	0.16	0.21	29/11/96
Dec					
Jan					
Feb	0.64	0.69	0.44	0.25	05/02/97
Mar	0.66	0.71	0.46	0.40	10/03/97
Apr	2.05	1.43	2.93	0.40	09/04/97
May	2.20	1.36	2.99	0.46	19/05/97
Jun	1.61	1.11	1.78	0.36	03/06/97

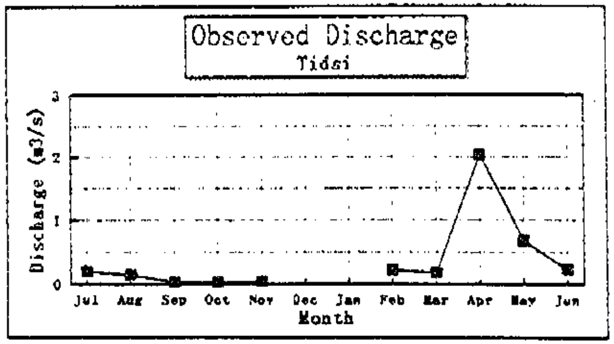




Tidsi 24 km2

	Mean velocity (m/s)	Flow area (m2)	Discharge (m3/s)	Water depth (m)	Date
Jul	0.63	0.32	0.20	0.18	17/07/98
Aug	0.53	0.28	0.15	0.15	02/08/96
Sep	0.24	0.14	0.03	0.10	06/09/96
Oct	0.07	0.36	0.03	0.18	03/10/96
Nov	0.11	0.41	0.04	0.11	30/11/96
Dec					
Jan					
Feb	0.23	0.95	0.22	0.31	06/02/97
Mar	0.13	1.39	0.18	0.39	12/03/97
Apr	1.30	1.57	2.03	0.35	10/04/97
May	0.47	1.45	0.68	0.40	20/05/97
Jun	0.28	0.88	0.24	0.32	04/06/97





Note: Observation made in March show a different trend: too high a water level for discharge of similar magnitude.
For Tidsi the velocity in March was smaller than expected, probably due to human interferance or observation error.
No observation was made for December and January

Figure 4.3-1 Observation Record – Installed Hydrometric Stations

Mean Water Level (average of AM & PM data) - Infag

unit:(m)

	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul
	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean
1		0.05	0.07	0.08	0.09	0.27	0.19	0.17	0.23	0.37		
2		0.05	0.07	0.08	0.09	0.27	0.19	0.17	0.26	0.37		
3		0.05	0.07	0.09	0.10	0.27	0.19	0.16	0.26	0.36		
4		0.05	0.07	0.09	0.11	0.27	0.18	0.16	0.25	0.35		
5		0.04	0.07	0.09	0.10	0.27	0.18	0.16	0.25	0.35		
6	0.04	0.04	0.08	0.09	0.08	0.26	0.18	0.16	0.25	0.35		
7	0.04	0.04	0.08	0.08	0.09	0.24	0.18	0.16	0.27	0.35		
8	0.04	0.05	0.08	0.08	0.10	0.24	0.18	0.16	0.32	0.34		
9	0.05	0.04	0.08	0.08	0.10	0.22	0.18	0.16	0.34	0.33		
10	0.05	0.05	0.08	0.08	0.11	0.22	0.18	0.16	0.34	0.31		
11	0.05	0.05	0.08	0.09	0.12	0.22	0.17	0.16	0.34	0.29		
12	0.06	0.05	0.08	0.08	0.12	0.22	0.17	0.15	0.32	0.26		
13	0.06	0.06	0.08	0.12	0.13	0.22	0.17	0.15	0.31	0.24		
14	0.06	0.06	0.08	0.12	0.14	0.21	0.17	0.15	0.31	0.24		
15	0.06	0.06	0.08	0.11	0.14	0.20	0.17	0.15	0.31	0.24		
16	0.07	0.06	0.08	0.12	0.14	0.19	0.17	0.15	0.31	0.24		
17	0.06	0.06	0.08	0.12	0.14	0.19	0.17	0.15	0.31	0.24		
18	0.06	0.06	0.08	0.12	0.14	0.19	0.16	0.15	0.31	0.24		
19	0.06	0.06	0.09	0.11	0.14	0.19	0.16	0.15	0.32	0.24		
20	0.06	0.06	0.08	0.12	0.14	0.19	0.16	0.15	0.40	0.25		
21	0.06	0.06	0.07	0.11	0.14	0.19	0.16	0.15	0.44	0.26		
22	0.06	0.05	0.08	0.11	0.15	0.19	0.16	0.15	0.44	0.26		
23	0.06	0.06	0.08	0.10	0.17	0.19	0.16	0.15	0.40	0.26		
24	0.06	0.07	0.09	0.09	0.20	0.19	0.16	0.16	0.38	0.26		
25	0.06	0.07	0.08	0.09	0.21	0.19	0.16	0.16	0.38	0.26		
26	0.06	0.07	0.08	0.09	0.22	0.19	0.16	0.16	0.39	0.26		
27	0.06	0.07	0.08	0.09	0.22	0.19	0.16	0.17	0.39	0.26		
28	0.06	0.07	0.09	0.09	0.23	0.19	0.16	0.17	0.38	0.26		
29	0.05	0.06	0.08	0.09	0.25	0.19		0.17	0.38	0.26		
30	0.05	0.07	0.08	0.08	0.25	0.19		0.18	0.37	0.25		
31	0.05		0.08		0.27	0.19		0.19		0.25		

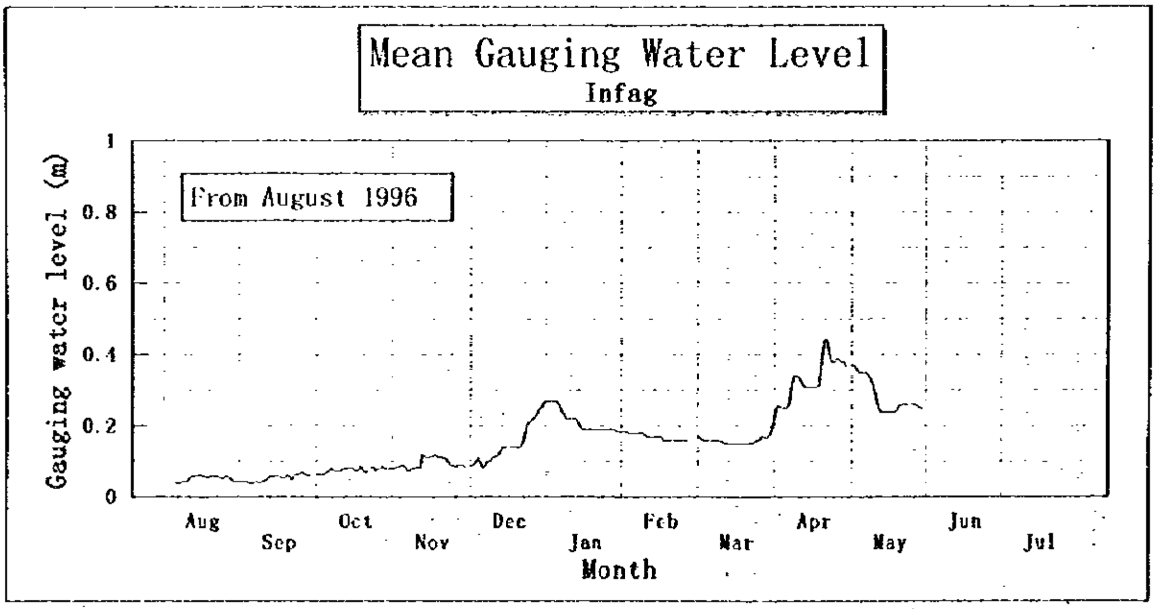


Figure 4.3-2 Mean Gauging Water Level – Infag

Mean Water Level (average of AM & PM data) - Arg

unit:(m)

	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul
	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean
1		0.44	0.35	0.37	0.41	0.48	0.47	0.31	0.58	0.39		
2		0.37	0.36	0.40	0.43	0.50	0.46	0.30	0.56	0.43		
3		0.41	0.39	0.40	0.41	0.52	0.46	0.28	0.51	0.45		
4		0.39	0.36	0.37	0.40	0.52	0.48	0.28	0.56	0.43		
5		0.40	0.38	0.35	0.42	0.53	0.47	0.28	0.61	0.41		
6		0.39	0.39	0.37	0.44	0.51	0.49	0.28	0.54	0.37		
7		0.38	0.39	0.35	0.44	0.52	0.49	0.28	0.71	0.36		
8		0.41	0.36	0.34	0.46	0.53	0.51	0.26	0.77	0.34		
9	0.33	0.41	0.37	0.36	0.44	0.51	0.51	0.29	0.74	0.35		
10	0.33	0.44	0.36	0.38	0.43	0.51	0.49	0.33	0.53	0.38		
11	0.33	0.45	0.38	0.36	0.42	0.51	0.46	0.36	0.53	0.41		
12	0.34	0.42	0.38	0.38	0.41	0.51	0.43	0.40	0.52	0.41		
13	0.34	0.40	0.41	0.41	0.41	0.51	0.42	0.41	0.50	0.43		
14	0.35	0.38	0.42	0.43	0.41	0.53	0.40	0.39	0.48	0.44		
15	0.36	0.34	0.44	0.41	0.43	0.53	0.38	0.37	0.46	0.38		
16	0.42	0.31	0.44	0.39	0.41	0.52	0.37	0.36	0.42	0.43		
17	0.42	0.34	0.41	0.40	0.41	0.54	0.39	0.35	0.44	0.45		
18	0.41	0.36	0.37	0.40	0.43	0.52	0.39	0.34	0.42	0.43		
19	0.39	0.34	0.35	0.38	0.44	0.53	0.37	0.32	0.44	0.44		
20	0.42	0.40	0.38	0.38	0.46	0.54	0.35	0.32	0.62	0.41		
21	0.38	0.39	0.34	0.38	0.45	0.54	0.34	0.31	0.64	0.37		
22	0.36	0.43	0.35	0.38	0.45	0.53	0.35	0.32	0.71	0.36		
23	0.41	0.45	0.36	0.40	0.47	0.53	0.35	0.32	0.74	0.37		
24	0.41	0.45	0.36	0.42	0.45	0.52	0.36	0.34	0.76	0.39		
25	0.39	0.37	0.33	0.40	0.43	0.52	0.36	0.35	0.76	0.38		
26	0.36	0.38	0.35	0.40	0.45	0.53	0.34	0.34	0.69	0.36		
27	0.41	0.38	0.35	0.37	0.47	0.52	0.33	0.35	0.60	0.35		
28	0.40	0.40	0.37	0.39	0.45	0.53	0.31	0.33	0.49	0.35		
29	0.39	0.40	0.37	0.42	0.45	0.52		0.35	0.47	0.38		
30	0.40	0.37	0.36	0.42	0.47	0.48		0.41	0.41	0.36		
31	0.41		0.34		0.47	0.48		0.52		0.36		

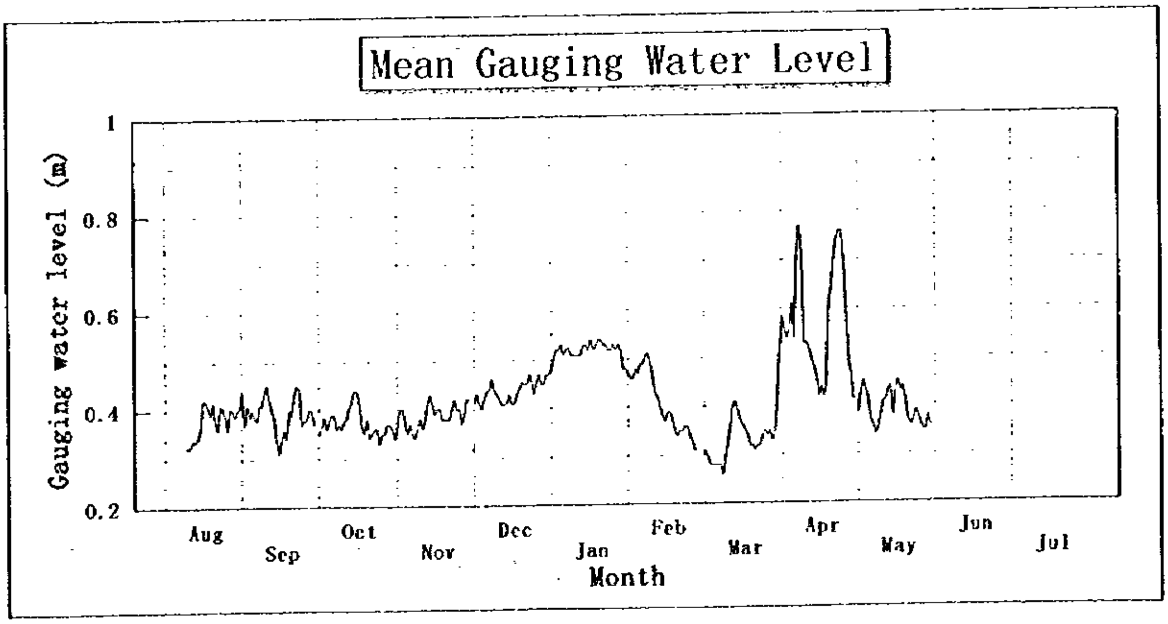


Figure 4.3-3 Mean Gauging Water Level – Arg

unit: (m)

	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul
	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean	Mean
1		0.29	0.30	0.30	0.31	0.35	0.31	0.38	0.41	0.39		
2		0.30	0.30	0.30	0.30	0.35	0.32	0.37	0.34	0.39		
3		0.31	0.30	0.31	0.28	0.35	0.33	0.37	0.30	0.38		
4		0.29	0.29	0.30	0.30	0.31	0.32	0.37	0.36	0.39		
5		0.29	0.30	0.30	0.38	0.29	0.33	0.35	0.41	0.39		
6		0.32	0.30	0.30	0.35	0.29	0.30	0.36	0.39	0.35		
7		0.30	0.30	0.30	0.35	0.28	0.33	0.36	0.41	0.36		
8	0.30	0.30	0.29	0.30	0.37	0.28	0.39	0.35	0.40	0.37		
9	0.30	0.30	0.29	0.31	0.35	0.27	0.35	0.35	0.39	0.39		
10	0.31	0.30	0.29	0.30	0.39	0.28	0.33	0.34	0.38	0.38		
11	0.29	0.32	0.30	0.31	0.41	0.27	0.32	0.34	0.38	0.36		
12	0.31	0.33	0.30	0.31	0.43	0.28	0.35	0.39	0.38	0.38		
13	0.32	0.33	0.30	0.32	0.43	0.27	0.37	0.39	0.40	0.39		
14	0.30	0.33	0.30	0.31	0.39	0.28	0.38	0.38	0.38	0.39		
15	0.30	0.33	0.31	0.36	0.40	0.28	0.37	0.39	0.38	0.40		
16	0.30	0.34	0.30	0.37	0.41	0.28	0.38	0.39	0.37	0.39		
17	0.29	0.33	0.30	0.34	0.41	0.28	0.38	0.39	0.38	0.39		
18	0.30	0.32	0.31	0.33	0.42	0.28	0.38	0.38	0.38	0.39		
19	0.27	0.31	0.30	0.33	0.40	0.31	0.38	0.39	0.37	0.39		
20	0.29	0.30	0.30	0.32	0.40	0.32	0.38	0.39	0.39	0.38		
21	0.29	0.30	0.30	0.32	0.38	0.33	0.37	0.39	0.40	0.39		
22	0.31	0.31	0.29	0.31	0.34	0.35	0.37	0.40	0.40	0.38		
23	0.31	0.31	0.30	0.31	0.34	0.36	0.38	0.39	0.40	0.39		
24	0.29	0.32	0.31	0.30	0.33	0.37	0.37	0.39	0.42	0.38		
25	0.27	0.32	0.30	0.31	0.31	0.39	0.38	0.38	0.40	0.36		
26	0.28	0.34	0.31	0.30	0.30	0.40	0.40	0.38	0.39	0.37		
27	0.30	0.30	0.30	0.31	0.29	0.40	0.38	0.40	0.38	0.36		
28	0.30	0.28	0.30	0.31	0.30	0.40	0.37	0.47	0.35	0.38		
29	0.30	0.29	0.30	0.25	0.30	0.40		0.47	0.35	0.39		
30	0.30	0.30	0.30	0.32	0.32	0.40		0.53	0.34	0.39		
31	0.30		0.30		0.34	0.40		0.57		0.38		

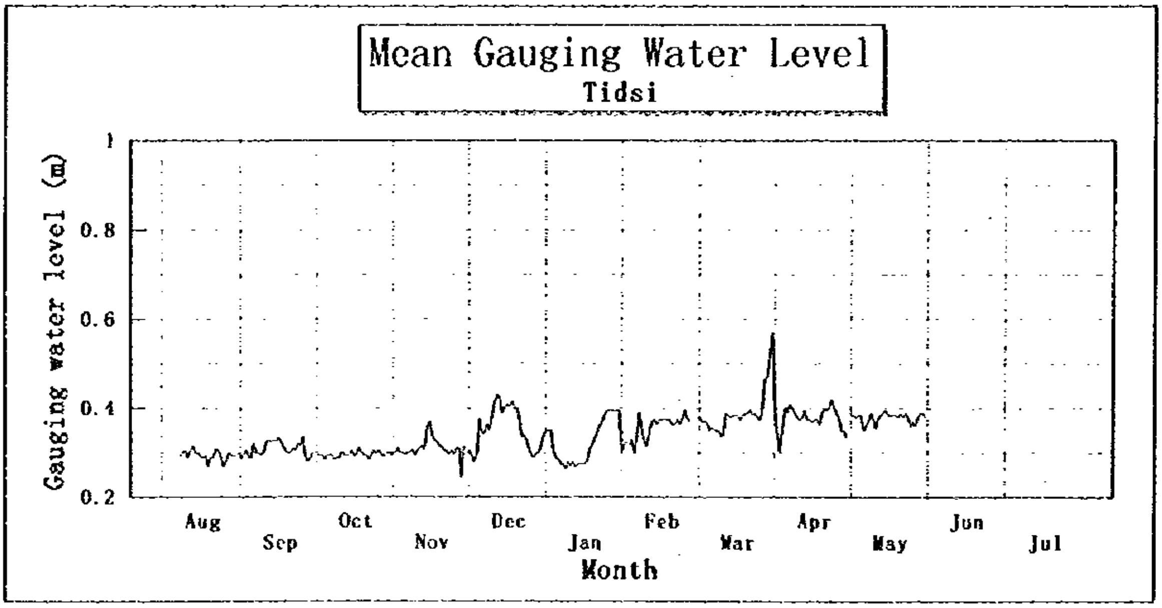
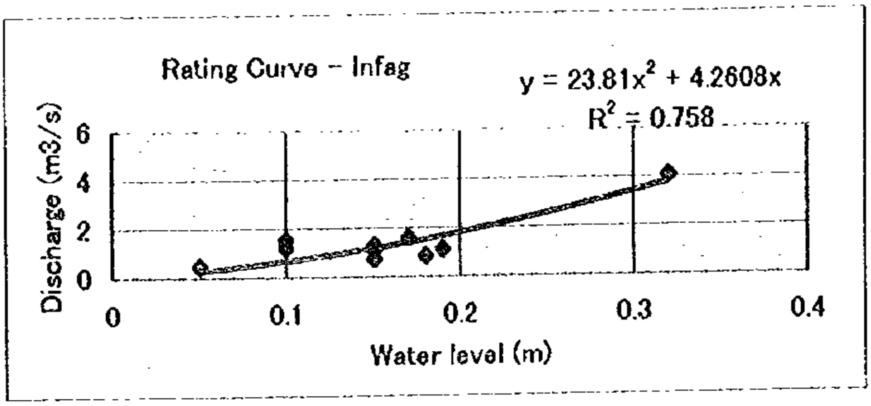
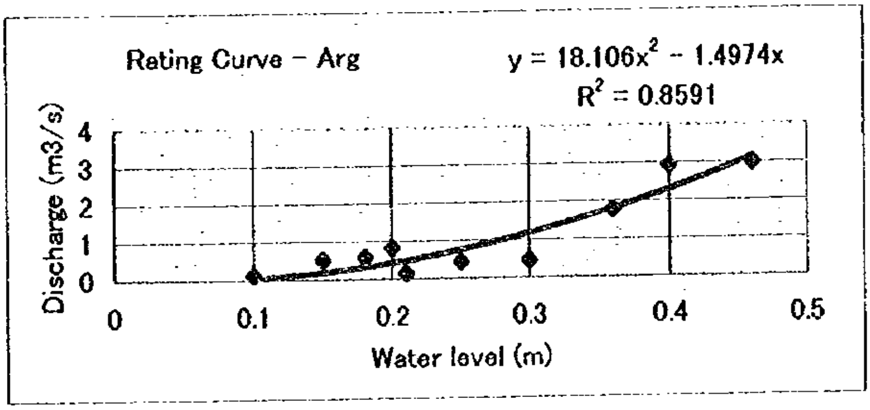


Figure 4.3-4 Mean Gauging Water Level -- Tidsi





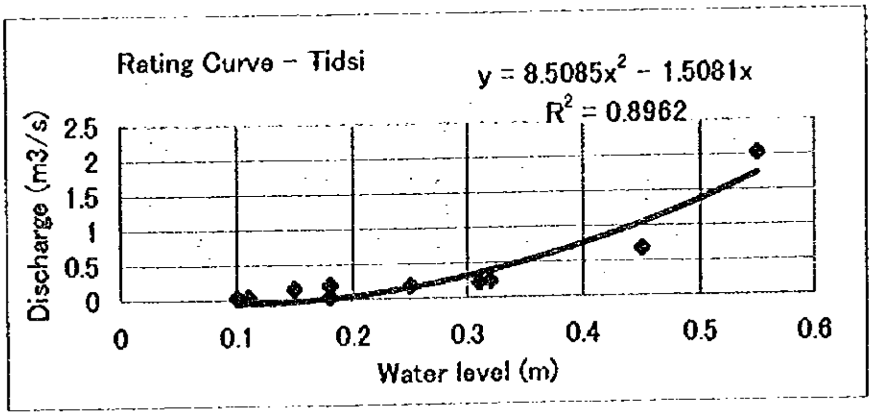
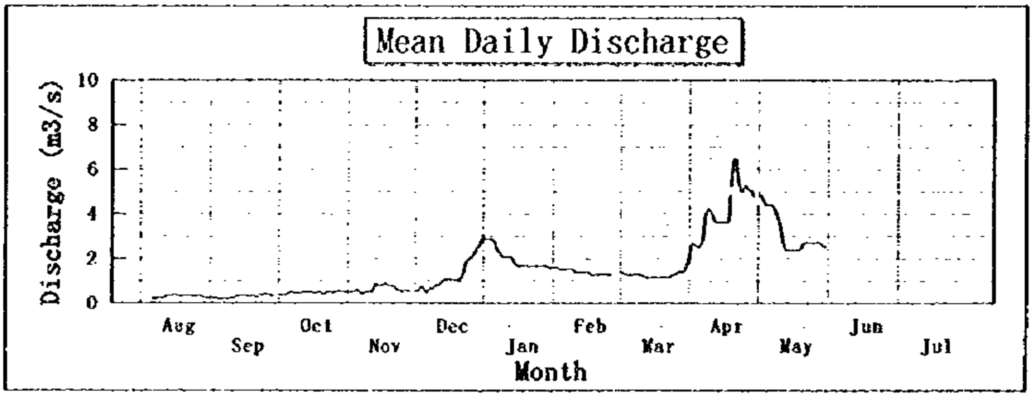


Figure 4.3-5 Rating Curve after Refinement – Installed Stations

Mean Daily Discharge - Infag Unit:(m3/s)

Year	1996					1997						
Month	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul
Day/1		0.240	0.378	0.493	0.576	2.886	1.603	1.412	2.164	4.836		
2		0.240	0.378	0.493	0.576	2.886	1.603	1.351	2.635	4.836		
3		0.240	0.378	0.534	0.620	2.886	1.603	1.291	2.635	4.620		
4		0.240	0.378	0.534	0.757	2.886	1.538	1.291	2.553	4.408		
5		0.209	0.415	0.576	0.620	2.886	1.538	1.291	2.553	4.408		
6	0.209	0.209	0.454	0.534	0.493	2.717	1.538	1.291	2.553	4.408		
7	0.209	0.209	0.493	0.454	0.576	2.394	1.538	1.291	2.801	4.408		
8	0.209	0.240	0.454	0.454	0.664	2.316	1.538	1.291	3.802	4.201		
9	0.240	0.209	0.454	0.493	0.664	2.090	1.538	1.291	4.201	3.999		
10	0.240	0.240	0.454	0.493	0.757	2.090	1.538	1.291	4.201	3.609		
11	0.240	0.240	0.454	0.534	0.805	2.090	1.412	1.232	4.100	3.148		
12	0.306	0.273	0.493	0.493	0.805	2.090	1.412	1.175	3.802	2.635		
13	0.341	0.306	0.493	0.851	0.905	2.090	1.412	1.175	3.609	2.394		
14	0.341	0.341	0.493	0.805	1.063	1.945	1.412	1.175	3.609	2.394		
15	0.341	0.341	0.493	0.757	1.063	1.736	1.412	1.175	3.609	2.394		
16	0.378	0.341	0.493	0.805	1.063	1.669	1.412	1.175	3.609	2.394		
17	0.341	0.341	0.454	0.805	1.063	1.669	1.412	1.175	3.609	2.394		
18	0.341	0.306	0.454	0.854	1.063	1.669	1.291	1.175	3.609	2.394		
19	0.306	0.306	0.534	0.757	1.063	1.669	1.291	1.175	3.705	2.394		
20	0.341	0.341	0.454	0.805	1.063	1.669	1.291	1.175	5.398	2.473		
21	0.341	0.341	0.415	0.757	1.063	1.669	1.291	1.175	6.484	2.717		
22	0.341	0.273	0.454	0.757	1.118	1.669	1.291	1.175	6.484	2.717		
23	0.341	0.341	0.493	0.664	1.351	1.669	1.291	1.175	5.514	2.717		
24	0.306	0.378	0.534	0.576	1.805	1.669	1.291	1.232	5.057	2.717		
25	0.306	0.378	0.454	0.576	1.945	1.669	1.291	1.291	5.057	2.717		
26	0.306	0.415	0.493	0.534	2.017	1.669	1.291	1.291	5.283	2.717		
27	0.341	0.378	0.493	0.576	2.090	1.669	1.291	1.412	5.170	2.717		
28	0.306	0.378	0.534	0.576	2.240	1.669	1.291	1.412	5.057	2.717		
29	0.273	0.341	0.493	0.576	2.473	1.669		1.351	5.057	2.635		
30	0.240	0.378	0.493	0.493	2.553	1.603		1.475	4.727	2.553		
31	0.240		0.493		2.886	1.603		1.669		2.553		



Design discharge (m3/s)

Q1	6.484
Q35	2.886
Q95	1.669
Q185	0.664
Q275	0.306
Q329	0.209
Q355	0.209
Q365	0.209

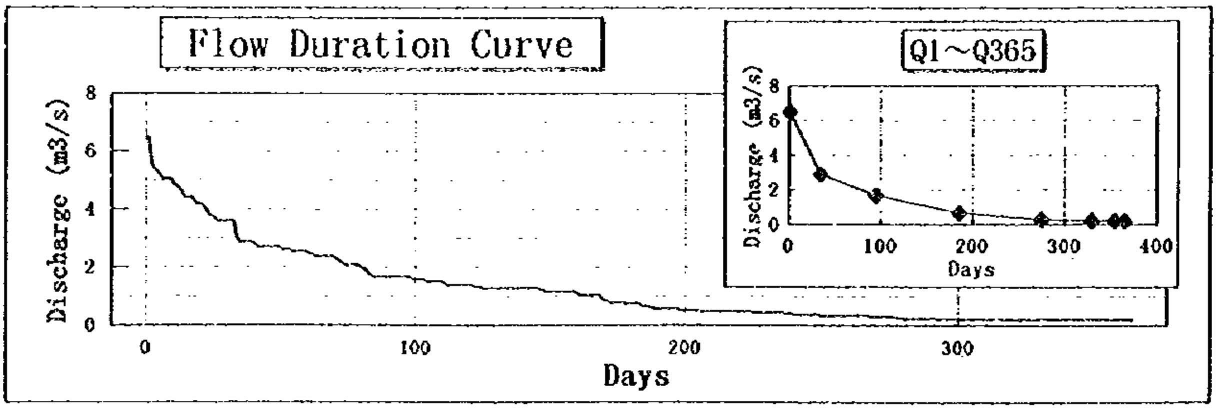
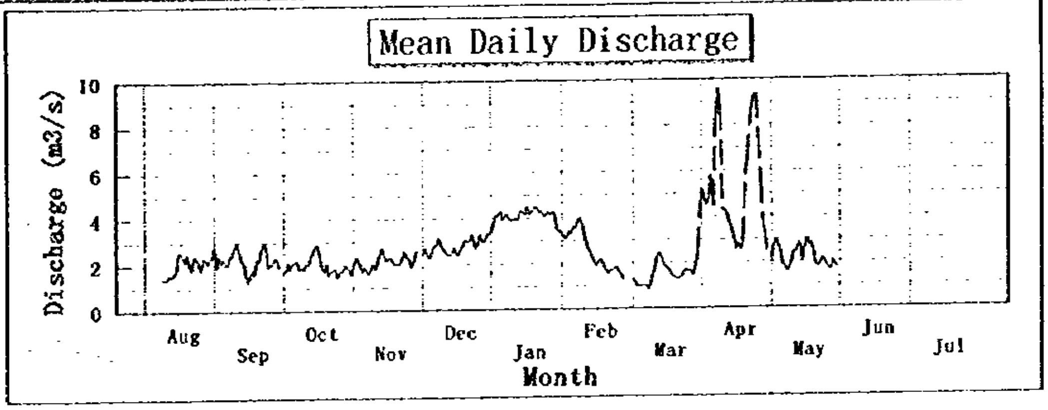


Figure 4.3-6 Mean Daily Discharge and Flow Duration Curve – Infag

Mean Daily Discharge - Arg

Unit:(m3/s)

Year	1996					1997						
Month	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul
Day/1		2.775	1.694	1.925	2.430	3.453	3.296	1.276	5.222	2.170		
2		1.925	1.807	2.298	2.704	3.778	3.142	1.180	4.839	2.704		
3		2.430	2.107	2.298	2.430	4.117	3.142	1.000	4.471	2.993		
4		2.107	1.807	1.925	2.298	4.117	3.453	1.000	4.839	2.704		
5		2.298	2.015	1.694	2.565	4.292	3.296	1.000	5.824	2.430		
6		2.107	2.170	1.925	2.775	3.916	3.614	1.000	4.471	1.925		
7		2.045	2.170	1.694	2.846	4.117	3.614	1.000	8.064	1.807		
8		2.430	1.807	1.584	3.142	4.204	3.916	0.835	9.582	1.584		
9	1.426	2.430	1.925	1.807	2.846	3.916	3.916	1.088	8.807	1.694		
10	1.426	2.846	1.807	2.045	2.704	3.916	3.614	1.478	4.292	2.045		
11	1.426	2.993	2.015	1.807	2.565	3.916	3.142	1.807	4.292	2.430		
12	1.584	2.565	2.045	2.045	2.430	3.916	2.704	2.298	4.117	2.430		
13	1.530	2.298	2.363	2.430	2.430	3.916	2.565	2.430	3.778	2.704		
14	1.638	2.045	2.565	2.704	2.430	4.292	2.298	2.170	3.453	2.846		
15	1.807	1.584	2.846	2.430	2.704	4.292	2.045	1.925	3.142	2.045		
16	2.565	1.276	2.846	2.170	2.430	4.117	1.925	1.807	2.565	2.704		
17	2.565	1.584	2.430	2.298	2.430	4.471	2.170	1.694	2.846	2.993		
18	2.430	1.807	1.925	2.298	2.704	4.117	2.170	1.584	2.565	2.704		
19	2.170	1.584	1.694	2.045	2.846	4.292	1.925	1.375	2.846	2.846		
20	2.497	2.298	2.045	2.015	3.067	4.471	1.694	1.375	6.032	2.430		
21	2.045	2.170	1.584	2.045	2.993	4.471	1.584	1.276	6.458	1.925		
22	1.807	2.704	1.694	2.015	2.993	4.292	1.694	1.375	8.064	1.807		
23	2.430	2.993	1.807	2.298	3.296	4.292	1.694	1.375	8.807	1.925		
24	2.363	2.919	1.807	2.565	2.993	4.117	1.807	1.584	9.320	2.170		
25	2.107	1.925	1.478	2.298	2.704	4.117	1.807	1.694	9.320	2.045		
26	1.807	2.045	1.694	2.298	2.993	4.292	1.584	1.584	7.587	1.807		
27	2.363	2.015	1.694	1.925	3.296	4.117	1.478	1.694	5.620	1.694		
28	2.298	2.298	1.925	2.170	2.993	4.292	1.276	1.478	3.614	1.694		
29	2.107	2.298	1.925	2.565	2.993	4.117		1.694	3.296	2.045		
30	2.298	1.925	1.807	2.565	3.296	3.453		2.430	2.430	1.807		
31	2.363		1.584		3.296	3.453		4.117		1.807		



Design discharge

(m3/s)

Q1	9.582
Q35	4.117
Q95	2.846
Q185	2.045
Q275	1.478
Q329	0.835
Q355	0.835
Q365	0.835

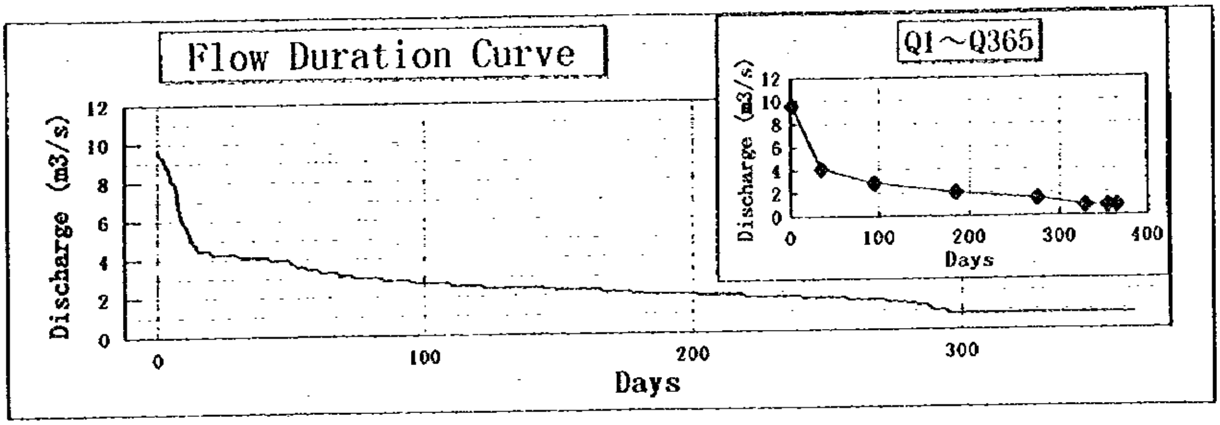
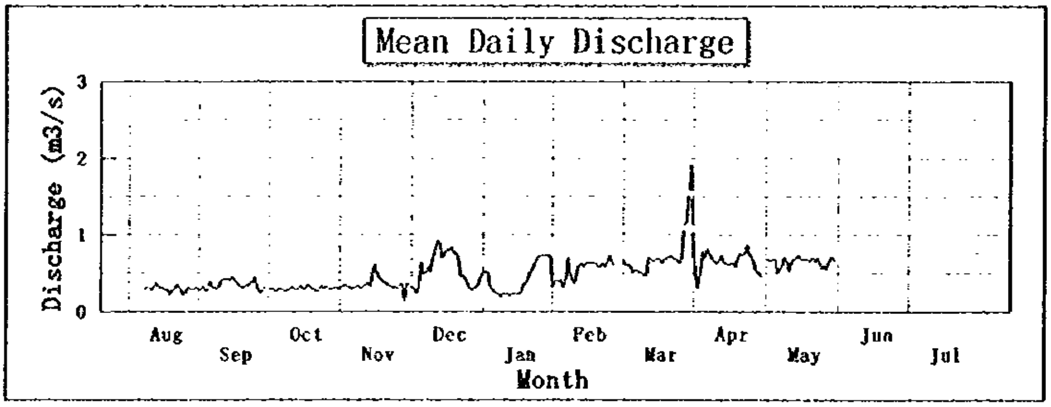


Figure 4.3-7 Mean Daily Discharge and Flow Duration Curve – Arg

Mean Daily Discharge - Tidsi　　Unit:(m3/s)

Year	1996					1997						
Month	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul
Day/1		0.278	0.296	0.313	0.332	0.514	0.332	0.656	0.812	0.681		
2		0.296	0.296	0.313	0.296	0.514	0.389	0.607	0.471	0.681		
3		0.332	0.296	0.350	0.245	0.514	0.409	0.607	0.313	0.656		
4		0.278	0.261	0.313	0.296	0.332	0.369	0.607	0.537	0.681		
5		0.278	0.296	0.313	0.631	0.278	0.429	0.514	0.785	0.681		
6		0.389	0.296	0.296	0.492	0.261	0.313	0.537	0.706	0.514		
7		0.313	0.296	0.313	0.492	0.229	0.429	0.537	0.812	0.560		
8	0.296	0.313	0.261	0.313	0.583	0.229	0.706	0.514	0.758	0.607		
9	0.313	0.296	0.278	0.332	0.514	0.198	0.514	0.514	0.706	0.706		
10	0.332	0.313	0.278	0.296	0.681	0.245	0.409	0.471	0.656	0.656		
11	0.278	0.389	0.313	0.332	0.812	0.213	0.369	0.471	0.631	0.537		
12	0.332	0.429	0.296	0.350	0.925	0.245	0.514	0.706	0.656	0.656		
13	0.369	0.409	0.296	0.369	0.896	0.213	0.607	0.681	0.732	0.681		
14	0.313	0.429	0.296	0.332	0.706	0.229	0.631	0.656	0.631	0.706		
15	0.313	0.409	0.332	0.537	0.758	0.229	0.583	0.681	0.631	0.732		
16	0.296	0.450	0.296	0.607	0.812	0.229	0.631	0.681	0.607	0.681		
17	0.278	0.409	0.296	0.471	0.785	0.229	0.631	0.681	0.631	0.681		
18	0.296	0.369	0.350	0.429	0.810	0.245	0.631	0.631	0.631	0.681		
19	0.213	0.332	0.313	0.409	0.758	0.350	0.631	0.681	0.583	0.681		
20	0.278	0.313	0.296	0.369	0.758	0.369	0.631	0.706	0.706	0.656		
21	0.278	0.313	0.296	0.369	0.656	0.429	0.583	0.706	0.758	0.706		
22	0.350	0.350	0.261	0.332	0.450	0.492	0.583	0.732	0.732	0.656		
23	0.332	0.332	0.296	0.332	0.471	0.560	0.631	0.681	0.758	0.706		
24	0.278	0.389	0.332	0.296	0.429	0.607	0.583	0.706	0.867	0.656		
25	0.213	0.369	0.296	0.332	0.350	0.681	0.631	0.656	0.758	0.560		
26	0.245	0.450	0.332	0.296	0.296	0.732	0.732	0.631	0.706	0.607		
27	0.313	0.296	0.296	0.350	0.278	0.732	0.656	0.732	0.631	0.560		
28	0.296	0.245	0.296	0.350	0.313	0.732	0.607	1.138	0.514	0.656		
29	0.296	0.261	0.296	0.141	0.313	0.732		1.171	0.514	0.706		
30	0.296	0.313	0.313	0.369	0.389	0.732		1.591	0.450	0.706		
31	0.313		0.296		0.471	0.732		1.905		0.656		



Design discharge (m3/s)

Q1	1.905
Q35	0.706
Q95	0.631
Q185	0.350
Q275	0.261
Q329	0.141
Q355	0.141
Q365	0.141

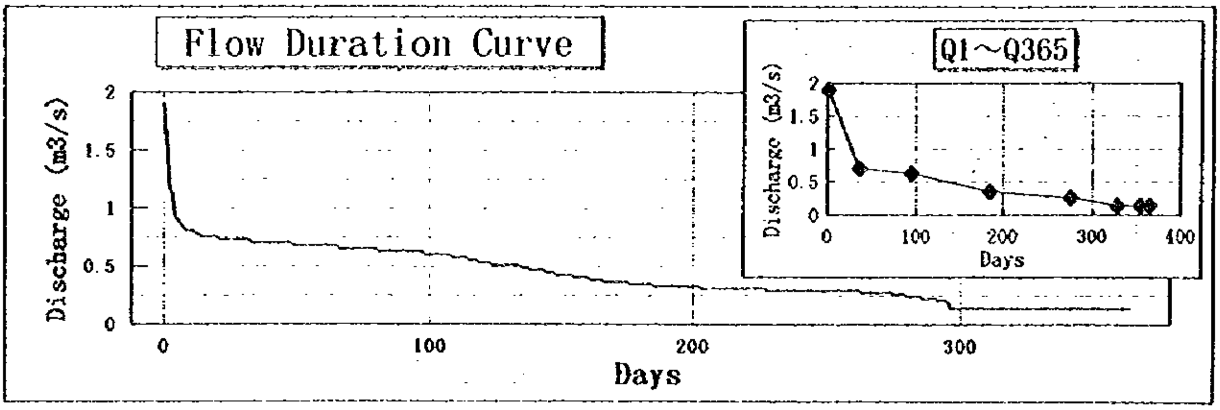


Figure 4.3-8 Mean Daily Discharge and Flow Duration Curve – Tidsi

Maximum daily rainfall - Existing Gauging Stations

unit (mm/day)

Taferiat		Tahanaout		Imin El Hammam		Aghbalou	
Year	rainfall	Year	rainfall	Year	rainfall	Year	rainfall
1981	30.0	1975	49.4	1971	38.1	1972	59.5
1982	58.6	1979	31.3	1975	26.5	1974	48.6
1984	38.7	1981	35.0	1976	26.8	1975	46.7
1985	28.3	1982	37.7	1977	21.5	1976	46.0
1990	34.0	1984	33.1	1978	35.1	1977	48.9
1991	69.3	1985	37.3	1979	25.0	1978	56.7
1992	21.8	1986	37.3	1980	30.5	1979	37.8
1993	31.3	1987	45.5	1981	28.4	1981	39.8
1995	68.0	1988	23.7	1982	65.6	1982	66.7
		1989	64.8	1983	32.5	1983	32.2
		1991	56.7	1984	32.3	1984	46.4
		1992	39.6	1985	28.0	1985	55.4
		1993	29.6	1987	26.7	1986	36.0
				1988	50.0	1987	57.1
				1989	53.5	1988	42.5
				1990	44.6	1989	67.2
				1991	32.9	1990	38.3
				1992	31.0	1991	71.6
				1993	26.0	1992	41.5
				1995	51.0	1993	22.7
						1994	61.9
						1995	67.8

Note: Year with data gap was omitted. Iguir N'Kouris was omitted due to lack of data.

Design Daily Rainfall- Existing Stations

Return Period	1/2	1/10	1/30	1/50	1/100
Taferiat	39.11	66.45	83.38	91.24	102.03
Aghbalou	47.96	69.08	80.85	86.06	93.02
Tahanaout	38.57	55.01	64.29	68.42	73.96
Imin El Hammam	32.55	49.96	62.28	68.34	76.98

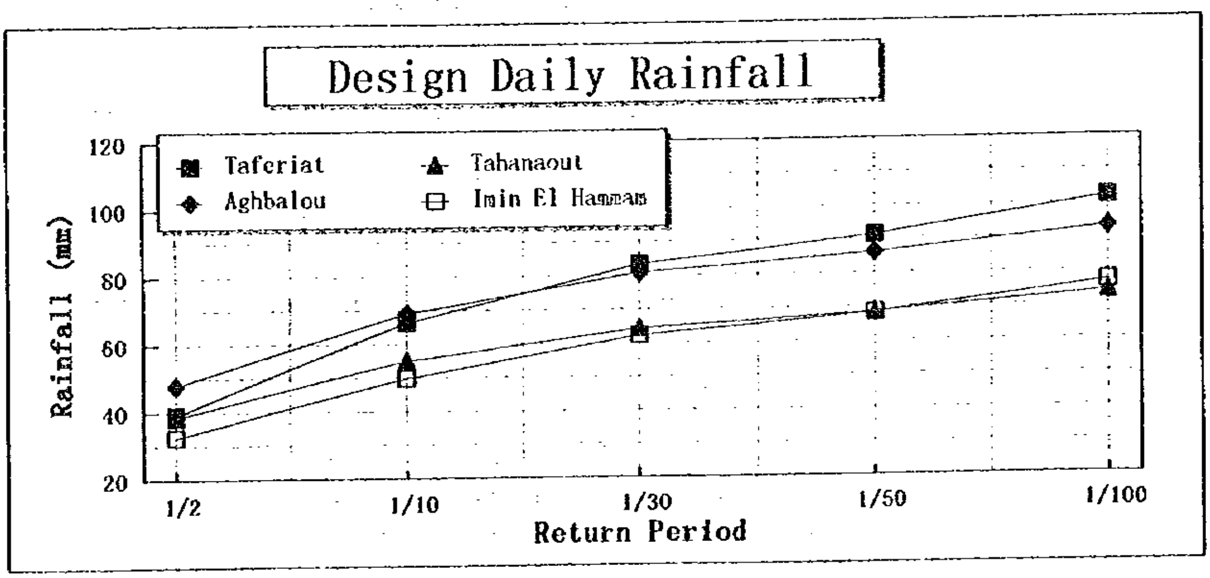


Figure 4.4-1 Design Daily Rainfall – Existing Stations

Table 4.4-1 Design Peak flood Discharge

unit:(m3/s)

Location	Adardour			Arg			Tidsi		
Area (km2)	23			48			24		
R.P.	1/10	1/50	1/100	1/10	1/50	1/100	1/10	1/50	1/100
Intake	139.4	190.7	214.9	343.3	427.0	461.6	248.8	341.6	382.0
Generation	145.5	199.0	224.2	356.9	443.9	479.8	276.8	380.1	425.0

Peak flood discharge per km2 unit:(m3/s/km2)

Location	Adardour			Arg			Tidsi		
R.P.	1/10	1/50	1/100	1/10	1/50	1/100	1/10	1/50	1/100
Intake	6.06	8.29	9.34	7.15	8.90	9.62	10.37	14.23	15.92
Generation	6.33	8.65	9.75	7.43	9.25	10.00	11.53	15.84	17.71

Estimation of flood discharge by Rational Formula

$Q = 0.2778 * f * r * A$

Q = Peak flood discharge (m3/s)
f = Runoff coefficient
r = Average rainfall intensity during flood concentration (mm/hr)
A = Catchment area (km2)

f= 0.9 for steep mountainous slopes

a) Flood concentration time T (hr)

$T = L / W$

L = Distance between drop points (km)
W = Flood concentration speed (km/hr), from b)

b) Flood concentration speed W (km/hr)

Bayern $W = 72 * (H/L)^{0.6}$ km/hr
Kraven $W = 3.5$ m/s or 12.6 km/hr for H/L > 1/100

H = Drop in elevation (km)
L = Distance between drop points (km)

c) Average rainfall intensity during flood concentration

Monobe equation $r = (R24 / 24) / (24 / T)^{(2/3)}$

r = Average rainfall intensity (mm/hr)
R24 = Design daily rainfall (mm/day)
T = Flood concentration time (hr), from a)

Table 4.4-2 Design Peak Flood Level

Design Peak Flood Discharge

unit: (m3/s)

Location	Adardor			Arg			Tidsi		
Area (km2)	23			48			24		
R. P.	1/10	1/50	1/100	1/10	1/50	1/100	1/10	1/50	1/100
Intake	139. 4	190. 7	214. 9	343. 3	427	461. 6	248. 8	341. 6	382
Generation	145. 5	199	224. 2	356. 9	443. 9	479. 8	276. 8	380. 1	425

Peak Flood Depth and Level - Intake Structure

y=a*x^b

Location	Adardor			Arg			Tidsi		
R. P.	1/10	1/50	1/100	1/10	1/50	1/100	1/10	1/50	1/100
a=	3. 8968			4. 6488			4. 6872		
b=	0. 4671			0. 4674			0. 4636		
Bed elevation	1, 769. 4			1, 573. 3			1, 724. 1		
Depth (m)	1. 55	1. 80	1. 90	2. 82	3. 12	3. 24	2. 46	2. 85	3. 00
Peak Flood (m)	1, 771. 0	1, 771. 2	1, 771. 3	1, 576. 1	1, 576. 4	1, 576. 5	1, 726. 6	1, 726. 9	1, 727. 1

Peak Flood Depth and Level - Intake Sites

y=a*x^b

Location	Adardor			Arg			Tidsi		
R. P.	1/10	1/50	1/100	1/10	1/50	1/100	1/10	1/50	1/100
a=	3. 8084			3. 8351			4. 521		
b=	0. 3918			0. 3638			0. 467		
Bed elevation	1, 766. 4			1, 573. 5			1, 722. 1		
Depth (m)	1. 76	1. 99	2. 09	2. 60	2. 81	2. 89	2. 36	2. 74	2. 88
Peak Flood (m)	1, 768. 2	1, 768. 4	1, 768. 5	1, 576. 1	1, 576. 3	1, 576. 4	1, 724. 5	1, 724. 8	1, 725. 0

Peak Flood Depth and Level - Generation sites

y=a*x^b

Location	Adardor			Arg			Tidsi		
R. P.	1/10	1/50	1/100	1/10	1/50	1/100	1/10	1/50	1/100
a=	3. 2062			3. 8205			4. 9156		
b=	0. 3665			0. 4029			0. 4812		
Bed elevation	1, 710. 1			1, 525. 7			1, 700. 3		
Depth (m)	1. 58	1. 77	1. 85	2. 52	2. 75	2. 84	2. 65	3. 09	3. 26
Peak Flood (m)	1, 711. 6	1, 711. 8	1, 711. 9	1, 528. 2	1, 528. 4	1, 528. 5	1, 702. 9	1, 703. 3	1, 703. 5

VOL. II CHAPTER 5

FORMULATION OF OPTIMUM GENERATING PLAN

CHAPTER 5 FORMULATION OF OPTIMUM GENERATING PLAN

5.1 Selection of Canal Route Alignment and Sites for Major Facilities

5.1.1 Basic Strategy for Selection

The 3 scheme sites selected for prefeasibility study are located in catchment areas ranging 23~48 km², which feature low annual average discharge of 0.25~0.57 m³/s. In addition, river gradient is not great, resulting in low available head vis a vis river size. In conjunction with this, existing canals divert river discharge for irrigation purposes making the ultimate power generating scale possible small at around 15~30 kW.

Generation method is to be by run-of-river. Selection of canal route alignment is constrained by inter-linkage with irrigation requirements. In this regard, the most cost-effective approach is to upgrade existing canal to function for the dual purposes of hydropower generation and irrigation.

The siting of other major facilities is likewise constrained. It is necessary to select intake and tailrace sites which minimize impact on existing irrigation systems. Taking into consideration topographical and geological features at the scheme sites, maximum effort is made to achieve the greatest possible head with minimal headrace canal length.

5.1.2 Adardour Scheme

Adardour scheme stands on river bed 1,770 m above sea level, with catchment area of 23 km² and average river gradient of 1:1.5. Annual average river flow is 0.25 m³/s. In order to plan a generating facility to meet the power demand forecast for the target villages for power supply, site surveys were carried out in terms of river flow, head, topography, geology, existing irrigation channel conditions, access road, difficulty / easiness of construction works, etc.. Consequently, optimum sites for intake facility, waterway route and powerhouse were selected. Topographic survey and geological survey (by surface exploration) were also conducted.

Particularly, regarding the intake facility site and the waterway route, the selection was done considering common use with the existing irrigation channel. The power house site was selected on the left bank (EL 1,716 m) in Adardour village considering necessary head on the basis of results of topographic and geological surveys.

The intake facility site was selected near the existing irrigation intake site at 1,770 m above sea level, and the intake weir (1.6m high) was planned to divert river water into the existing irrigation channel by the new intake on the left bank.

The waterway utilizes the existing irrigation water channel along the left bank by widening and improvement of the channel. The total length of the waterway is 685 m including partially newly constructed section (350 m long). Particularly, along the canal route, several outlets to supply irrigation water in the dry months will be established for effective water use and to function as spillways settling basins when during the high water season.

The head tank site at the end of the waterway was selected on the upside of the penstock and power house sites. The head tank size was designed on the wide side (1.5m wide, 10.0m long, 2.0m high) judging from its important role for regulating of the plant discharge. The site selected is flat, at 1,756m above sea level in a location of hard rock foundation.

The penstock and power house sites need to be located on stable hard foundation. As the said sites will function as an important base for O&M works after the schemes begin operation, the sites were selected at suitable locations below the headtank site. Tail water is planned not to discharge to the river below the outlet, but rather to discharge into the upper side of the existing irrigation weir through a connecting tailrace for irrigation water intake.

5.1.3 Arg Scheme

Arg scheme stands on the river bed at 1,573 m above sea level, with catchment area of 48km^2 and average river gradient of 1:20. Annual average river flow is 0.57m^3/s. In order to plan a generating facility meet the power demand forecast for the target villages for power supply, site surveys were carried out in terms of river flow, head, topography, geology, existing irrigation channel conditions, access road, difficulty / easiness of construction works, etc.. Consequently, optimum sites for intake facility, waterway route and power house were selected. Topographic survey and geological survey (by surface exploration) were also conducted.

Particularly, regarding the intake facility site and the waterway route, the selection was done considering common use with the existing irrigation channel. The power house site was selected on the left bank (EL 1,532 m) downstream of Arg village considering necessary head on the basis of results of topographic and geological surveys.

The intake facility site was selected near the existing irrigation intake site at 1,573m above sea level, and the intake weir (1.85m high) was planned to intake river water into the existing irrigation channel by the new intake on the left bank.

The waterway utilizes the existing irrigation water channel by widening and improvement of the channel. The total length of the waterway is of about 1,175 m including partially newly constructed section (near the head tank). Particularly, along the canal route, several outlets to supply irrigation water in the dry months will be established for effective water use and to function as spillways settling basins when during the high water season.

The head tank site at the end of the waterway was selected on the upside of the penstock and power house sites. The head tank size was designed on the wide side (2.0m wide, 10.0m long, 2.0m high) judging from its important role for regulating of the plant discharge. The site selected is flat, at 1,560 m above sea level in a location of hard rock foundation.

The penstock and power house sites need to be located on stable hard foundation. As the said sites will function as an important base for O&M works after the schemes begin operation, the sites were selected at suitable locations below the headtank site. Tailwater is planned for discharge into the river directly, due to no need for irrigation water supply to nearby areas.

5.1.4 Tidsi Scheme

Tidsi scheme stands on the river bed at 1,723 m above sea level, with catchment area of 24 km^2 and average river gradient of 1:25. Annual average river flow is 0.39m^3/s. In order to plan a generating facility meet the power demand forecast for the target villages for power supply, site surveys were carried out in terms of river flow, head, topography, geology, existing irrigation channel conditions, access road, difficulty / easiness of construction works, etc.. Consequently, optimum sites for intake facility, waterway route and power house were selected. Topographic survey and geological survey (by surface exploration) were also conducted.

Particularly, regarding the intake facility site and the waterway route, the selection was done considering common use with the existing irrigation channel. The power house site was selected on the right bank down stream (EL 1,703 m) of Tidsi village considering necessary head on the basis of the results of topographic and geological surveys.

The intake facility site was selected near the existing irrigation intake site (left bank) 1,703m above sea level, and the intake weir (2.15m high) was planned to divert river discharge into the existing irrigation channel by the new intake on the right bank. In the case of this intake weir, intake for existing irrigation water on the left bank was is also planned for effective utilization of river discharge for both power generation and irrigation purposes.

A headrace canal waterway about 750 m long will be newly constructed on the right bank, of open channel type with gradient of 1:400. Particularly, along the canal route, several outlets to supply irrigation water in the dry months will be established for effective water use and to function as spillways settling basins when during the high water season.

The head tank site at the end of the waterway was selected on the upside of the penstock and power house sites. The head tank size was designed on the wide side (1.5m wide, 10.0m long, 2.0m high) judging from its important role for regulating of the plant discharge. The site selected is flat, at 1,722 m above sea level in a location of hard rock foundation.

The penstock and power house sites need to be located on stable hard foundation. As the said sites will function as an important base for O&M works after the schemes begin operation, the sites were selected at suitable locations below the headtank site.

Tailwater is planned for discharge into the river directly, due to no need for irrigation water supply to nearby areas.

5.2 Transmission and Distribution Plan

5.2.1 Basic Plan Concept

The maximum distance from the power station site to the target villages is 5 km. Since low voltage transmission would result in a large transmission loss, generator voltage of 440 V is to be stepped up to 3.3 kV for conveyance to the villages at which point voltage would then be stepped down to 400 V / 220 V by pole mounted transformers in the villages for power supply to each household.

Transmission power poles are to be wood with the exception of villages with extremely poor access, where in such cases out of consideration of transport weight, assembly type, galvanized steel transmission poles will be adopted.

Standard power pole interval is to be 35~50 m, with this standard to be modified where necessary in light of actual in-situ topographical and meteorological conditions. Standard power pole height is to be 9 m (standard height prevailing in Morocco at present is 8~10.5 m).

In the case of distribution poles, the standard adopted is 1 pole per 5 homes, and these are to be laid out with consideration to power utilization by schools, mosques, etc. as well. As these poles will be mounted with transformers to step down voltage from 3.3 kV to 400 V / 220 V, it is recommended that they have a strength of 300 kg/cm².

Standard for transmission line cable is to be 34 mm² diameter, with unit resistance under 0.899 Ω/km. However, in cases where a high transmission loss may be anticipated, cable diameter will be selected from among the ONE standard of 75 mm² and 148 mm². It is recommended that cable material be either aluminum or steel, with strength of 1,140 kg/cm².

Transformer equipment to be mounted on distribution line poles is to feature manual switch gear, with 3.3 kV primary side and 400 V / 220 V secondary side for power delivery to the individual home.

Transmission line is designed to meet power demand to the year 2010, while distribution line is designed to meet power supply demand in the target year for completion of the Master Plan, i.e. the year 2000.

An outline of the transmission and distribution plan is indicated in Figure 5.2-1.

5.2.2 Adardour Scheme

The Adardour power station will be capable of maximum output of 26 kW and guaranteed output of 10.1 kW. Power is to be supplied from the station to 4 villages totaling 168 households (comprising 42 homes in Adardour, 32 in Tinezriosire, 32 in Tamalout, and 62 in Kettou).

The distance from the power station to Adardour village (42 households) is within a range of 50 m, which can be serviced by 2 transmission poles and 12 distribution poles, for a total of 14 power poles. One unit of distribution pole mounted transformer is to be installed. Power line length is to be 50 m of transmission line and 645 m of distribution line (including that to service elementary school, etc.)

In the case of Tinesroisire, a 750 m transmission line will provide power to 32 households. Power poles would total 26, comprising 14 transmission poles and 12 distribution poles. One unit of distribution pole mounted transformer is to be installed. Total distribution line length is 486 m.

The 32 households in Tamalout would be serviced by a transmission line branching off from the one to Tinezriosire. Transmission length from the power station is 250 m. Power poles would total 15, comprising 5 transmission poles and 10 distribution poles. One unit of distribution pole mounted transformer is to be installed. Total distribution line length is 480 m.

Transmission to the 62 households in Kettou would be by a 500 m extension of the line to Tenezriosire. Power poles would total 29, comprising 10 transmission poles and 19 distribution poles. One unit of distribution pole mounted transformer is to be installed. Total distribution line length is 1,055 m.

A summary of equipment and material required for the transmission and distribution plan from Adardour power station is as follows:

Power poles:	84 nos.
Pole mounted transformer (15 kVA):	4 nos.
Distribution line cable (34 mm²):	2,666 m
Transmission line cable (25 mm²):	1,550 m

5.2.3 Arg Scheme

The Arg power station will be capable of maximum output of 30 kW and guaranteed output of 15.4 kW. Power is to be supplied from the station to 3 villages totaling 205 households (comprising 79 homes in Arg, 58 in Amsakrou, and 68 in Ikiss).

In the case of Arg village, hamlets are located along the river with transmission and distribution from the power station overlapping. Power poles would total 55, with 2 units of distribution pole mounted transformer to be installed. Total transmission line length is 1,400 and total distribution line length is 1,150 m.

Amsakrou (58 households) is located 1,500 m from Arg village. Power poles would total 45, comprising 30 transmission poles and 15 distribution poles. One unit of distribution pole mounted transformer is to be installed. Total distribution line length is 920 m.

Transmission to Ikiss (68 households) would be by a 1,500 m extension of the line to Amsakrou. Power poles would total 51, with 1 unit of distribution pole mounted transformer to be installed. Total distribution line length is 1,100 m.

A summary of equipment and material required for the transmission and distribution plan from Arg power station is as follows:

Power poles:	151 nos.
Pole mounted transformer (15 kVA):	4 nos.
Distribution line cable (34 mm²):	3,170 m
Transmission line cable (25 mm²):	4,400 m

5.2.4 Tidsi Scheme

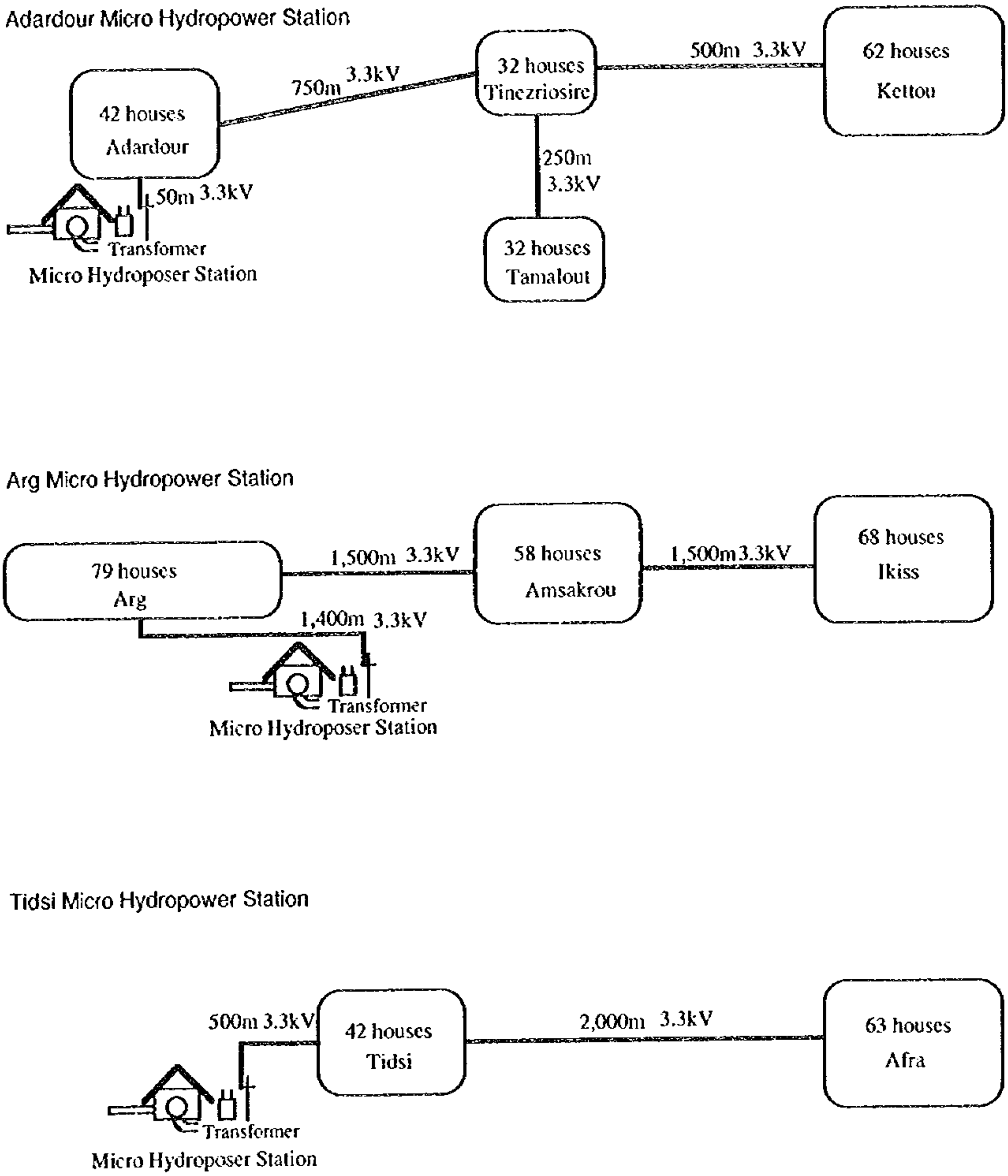
The Tidsi power station will be capable of maximum output of 15 kW and guaranteed output of 4.2 kW. Power is to be supplied from the station to 2 villages totaling 105 households (comprising 42 homes in Tidsi, and 63 in Afra).

Afra village was included under the Project in light of the fact that (i) it is transited by Tidsi villagers in the course of their day to day activities, (ii) it will be a transit point for material and equipment in the course of Project implementation, and (iii) Afra villagers are desirous of electrification under the Project. Tidsi village itself is located 500 m from the power station, and will require 22 transmission and distribution power poles, 1 unit of distribution pole mounted transformer, and 550 m of distribution line.

Afra is located 2,000 m from Tidsi village, and comprises 63 homes to be electrified. Power poles would total 55, comprising 40 transmission poles and 15 distribution poles. One unit of distribution pole mounted transformer is to be installed. Total distribution line length is 800 m.

A summary of equipment and material required for the transmission and distribution plan from Tidsi power station is as follows:

Power poles:	77 nos.
Pole mounted transformer (15 kVA):	2 nos.
Distribution line cable (34 mm²):	1,350 m
Transmission line cable (25 mm²):	2,500 m



*Numbers of household indicates as of 2000.

Figure 5.2-1
Conceptual Diagram for Transmission and Distribution Planning

VOL.II CHAPTER 6
PREFEASIBULITY LEVEL DESIGN

CHAPTER 6 PREFEASIBILITY LEVEL DESIGN

6.1 Civil Facilities

The generation scale of the 3 micro-hydropower stations are mini size between a range of 15 kW and 30 kW, and each site is subject to similar conditions of river flow, river gradient, topography, etc. Therefore, the basic design conception for each site is the same. Following is the basic concept concerning the prefeasibility level design.

(1) Intake Facility

In the survey area, there are many existing irrigation intake weirs. Most of these are made from cobble, and are susceptible to damage by flood. In this light, a reinforced concrete with reverse T type weir was applied under this Study. The weir does not stand on hard basement rock; however, the lower side of the weir is to be protected by cobble with mortar finish, and the upper side only by cobble. Intake structure is planned to adjoin the weir, and the material is reinforced concrete.

(2) Waterway

Owing to the fact that each waterway of for the 3 schemes is to serve both for generation and irrigation, the waterway should have a discharge capacity that takes into account irrigation water. The present utilized irrigation water is estimated at a maximum of about 0.05m^3/s.

The waterway is to be open channel type, and construction is to be cobble with interior mortar finishing. Cobble material is to be obtained near by.

(3) Headtank

The capacity of the headtank is planned to have about 2 minutes discharge volume, considering site conditions and operation conditions. The design conditions are as follows.

		Adardour	Arg	Tidsi
Maximum discharge	(m^3/s)	0.11	0.18	0.15
Active volume	(m^3)	15.0	20.0	15.0
Regulating time	(min)	2..2	1.8	1.8

(4) Penstock

Steel pipe, FRP and concrete pipe are possible materials to be considered for penstock. Of these, FRP is the most expensive and joint connecting work needs a sophisticated level of expertise. Although concrete pipe is inexpensive, transportation and installation of the same in mountainous area is considered difficult. Moreover, subsidence and cracking of joints are causes of water leakage. Accordingly, steal pipe was adopted under this Study.

Most of the penstock alignment is to be installed under the ground surface to avoid damage by rock fall, and penstock surface is to be protected by coating with anti-rust paint.

Concrete anchor blocks on solid foundation will stabilize the penstock.

(5) Powerhouse Building

The power house building is to be constructed of concrete block to be fabricated at the site. Building structure is to be such as to protect against rain, snow, rock fall, etc. A stilling basin is to be established below the turbine to function as a water cushion, and also to be utilized as a discharge chamber for dummy load.

(6) Tailrace

Tailrace is to be open channel type, constructed of cobble with interior mortar finishing.

6.2 Generating Equipment

6.2.1 Selection of Turbine

In selecting the turbine type, is necessary to consider the specific characteristics of each type, range of equipment application, operational efficiency, etc.

Types of turbine include the pelton class for high head, the francis class for intermediate head and the propeller class (kaplan, valve, etc.) for low head. In addition, specifically in the case of small hydropower schemes, there are the cross flow and turgo impulse class turbines (the latter often used where high head is available). Selection of turbine rpms is done on the basis of specific speed, and in this case type of turbine to be considered is limited.

Under the subject schemes, effective head is 15~37 m and generating discharge is 0.11~0.18 m^3/s. Specific speed for rpms of 1,000 will be 60~110 (rpm/kW/m). On the basis of this criteria, the cross flow type turbine is to be adopted.

Given the generating scheme type, fluctuation in head will be small; however, in light of the fact that scheme operation will be required during months of low river discharge as well in the course of the annual power supply plan, there will be a dramatic fluctuation in generating discharge. In the case of the conventional cross flow turbine, guide vanes are generally separated and generation is performed by one guide vane side in order to upgrade operational efficiency when generating discharge is low. Since maximum generating discharge is small to begin with under the subject Project, however, with narrow turbine runner and short guide vanes, the adoption of separate guide vanes is considered impractical.

Generally, in the case where separate guide vanes cannot be adopted in the case of the cross flow turbine, turbine efficiency radically drops when actual generating discharge drops below 30% of the maximum design discharge. In order to compensate for this, and with consideration to cost effectiveness, a small discharge reverse running pump turbine is to be installed separately for power generation during the low water season.

(1) Adardour Power Station

Given the river flow characteristics at the site, the approximate 275 day discharge (0.11 m^3/s) out of annual discharge is adopted at the maximum generating discharge, with firm discharge assumed at 0.043 m^3/s (90% of the annual discharge). On this basis, generation in the low water season utilizes less than 40% of maximum generating discharge, resulting in a drop in turbine efficiency. In order to improve turbine efficiency at such time, an inexpensive, reverse running pump turbine unit is to be separately installed to achieve efficient power output during the low water season. In the case of the Adardour scheme area, there is no other recourse to energy source and in this regard ensuring adequate power supply during the low water season becomes extremely important.

Since the time of operation of the reverse running pump type and the cross flow turbine do not coincide, there is no pressing need for the present to effect parallel operation of the two units. However, it is planned to install a synchronous operating system in consideration of the possible need to expand generating capacity in the future, and the case of eventual connection of the scheme to the existing grid.

(2) Arg Power Station

Maximum generating discharge is 0.18 m^3/s and firm discharge is 0.097 m^3/s. Generating discharge ratio is thus 54%, with little fluctuation in said discharge. Since this will not result in a significant drop in turbine efficiency, 1 unit of cross flow turbine is to be adopted under the scheme.

(3) Tidsi Power Station

Maximum generating discharge is 0.15 m^3/s and firm discharge is 0.044 m^3/s, which is a ratio of under 30%. A drop in turbine efficiency can therefore be anticipated. Accordingly, as in the case of the Adardour scheme, one unit of cross flow turbine is to be adopted as the prime equipment, to be supported by 1 unit of reverse running pump type in light of little head fluctuation and limited operation range under low load generating conditions.

6.2.2 Selection of Generating Equipment

In selection of generator, the synchronous type and induction type were considered. The induction type is more economical, as inexpensive, small motor equipment commonly available on the market can be utilized, particularly in the case of micro-hydropower generating schemes. However, an important difference between the induction type and synchronous type generator is that in the case of the former power factor adjustment is not possible. In the case of the latter, independent operation is facilitated by the fact that it is equipped with and independent excitation system. In light of the foregoing and the fact that power utilization is almost entirely for illumination load, the synchronous type generator has been selected despite higher cost due to the fact that power factor adjustment is possible.

6.2.3 Selection of Control System

The control system for hydropower generating facilities comprises turbine · generator start up, shut down and other protective devices. It is also a standard procedure to install a governor to maintain stable turbine · generator rpms and frequency. However, in the case of the subject Project where power output is small, generation frequency is to be stabilized by adjusting surplus power via resistance capacity. This is referred to as the "dummy load governor method".

6.2.4 Consideration of Hybrid Generation

The hybrid generation method comprises a combination of micro-hydropower and PV generation (battery charge). In order to improve the operating efficiency of the micro-hydropower facility, surplus power during off-peak hours is battery stored for subsequent use when such energy is required.

Under the subject Project, study was made of the possibility of installing battery, inverter and converter equipment appropriate for each scheme with a capacity of power supply for four hours each day over a 10 day period. As a result, in light of the need to adjust transmission voltage with the generator, the cost of installing the battery, inverter and converter equipment becomes larger than that for the generator itself, and in this regard is considered impractical for power supply for general lighting purposes.

6.3 Design Features

In line with the basic concept set out in Section 6.1 and 6.2, prefeasibility level design was carried out on the basis of topographical, geological and meteorological data. Design features are summarized in Table 6.3-1. Electrical equipment design is presented in Figure 6.3.1~6.3.6. Civil facilities design is attached at the end of this volume.

Table 6.3-1 Design Features of the Micro-hydropower Schemes (1/2)

Specification \ Power station	Unit	Adardour	Arg	Tidsi
Cercle		Amizmiz	Asni	Ait Ourir
Commune rurale		Anougal	Asni	Tighdouine Zerkten
Dour		Adardour	Arg & others	Tidsi, Afra
River		Amizmiz	Rhenaya	Zat
Tributary		Anougal	Imenane	Afoughal
Catchment area	km^2	23	48	24
Installed output	kW	26.0	30.0	15.0
Dependable output	kW	10.0	15.4	4.2
Maximum discharge	m^3/s	0.11	0.18	0.15
Dependable discharge	m^3/s	0.043	0.097	0.044
Intake water level	m	1,769.0	1,574.0	1,724.5
Headtank water level	m	1,757.0	1,561.0	1,722.0
Turbine center level	m	1,719.0	1,534.5	1,706.0
Gross head	m	38.0	26.5	16.0
Effetive head	m	37.0	25.0	15.0
Synthetic efficienfy	%	66	70	69
Generated energy	kWh	56,914	73,648	22,203
Intake weir				
Type		Reverse T type	Reverse T type	Reverse T type
Material		and cobble	and cobble	and cobble
Height	m	1.65	1.85	2.15
Crest length	m	16.0	15.0	14.0
Intake				
Type		Open channel	Open channel	Open channel
Size (B x L)	m	1.5 x 6.0	1.5 x 6.5	1.5 x 6.5
Gate (B x L)	m	1.0 x 1.0	1.0 x 1.0	1.0 x 1.0
Screen	unit	1	1	1
Waterway				
Type		Open channel	Open channel	Open channel
Length	m	685	1,175	750
Size (B x H)	m	0.6 x (0.6~1.0)	0.6 x (0.6~1.0)	0.6 x (0.6~1.0)
Gradient	%	1:100 ~ 1:300	1:100 ~ 1:300	1:400
Headtank				
Type		Open channel	Open channel	Open channel
Size (B x H x L)	m	1.5 x (1.5~2.0) x 10.0	2.0 x (1.5~2.0) x 10.0	1.5 x (1.5~2.0) x 10.0
Screen	unit	1	1	1
Penstock				
Type		Buried type	Buried type	Buried type
Material		Steel pipe	Steel pipe	Steel pipe
Length	m	76.4	84.0	33.8
Diameter	m	0.30	0.35	0.35
Tailrace				
Type		Open channel	Open channel	Open channel
Length	m	100	40	20
Size (B x H)	m	0.6 x (0.6 ~ 1.25)	1.0 x 1.0	0.6 x (0.6 ~ 1.0)
Power house buuilding				
Type		Above ground	Above ground	Above ground
Material		Concrete block	Concrete block	Concrete block
Size (B x L x H)	m	3.0 x 4.0 x 2.5	3.0 x 4.0 x 2.5	3.0 x 4.0 x 2.5

Table 6.3-1 Design Features of the Micro-hydropower Schemes (2/2)

Power station / Specification	Unit	Adardour	Arg	Tidsi
No.1 generation plant				
Turbine				
Type		Cross-flow	Cross-flow	Cross-flow
Output	kW	26	30	15
Effective head	m	37.0	25.0	15.0
Plant discharge	m^3/s	0.11	0.18	0.15
Revolutions	rpm	750	750	750
Generator				
Type		3 phase syncronous	3 phase syncronous	3 phase syncronous
Output	kVA	32.5	37.5	18.8
Voltage	V	440	440	440
Frequency	Hz	50	50	50
Revoltions	rpm	750	750	750
No.2 generation plant				
Turbine				
Type		Pump-reversal	-----	Pump-reversal
Output	kW	10.1	-----	5.0
Effective head	m	37.0	-----	15.0
Plant discharge	m^3/s	0.043	-----	0.044
Revolutions	rpm	1,000	-----	1,000
Generator				
Type		3 phase syncronous	-----	3 phase syncronous
Capacity	kVA	12.6	-----	6.3
Voltage	V	440	-----	440
Power factor		0.8	-----	0.8
Frequency	Hz	50	-----	50
Revoltions	rpm	1,000	-----	1,000
Transformer				
Type		3 phase out down	3 phase out down	3 phase out down
Voltage	V	440V / 3,300V	440V / 3,300V	440V / 3,300V
Capacity	kVA	39	45	30
Transmission line				
Line voltage	V	3,300	3,300	3,300
Wire length	m	1,550	4,400	2,500
Type of wire		$25mm^2$ aluminum	$25mm^2$ aluminum	$25mm^2$ aluminum
Distribution line				
Line voltage	V	220	220	220
Cable length	m	2,666	3,170	1,350
Type of cable		3 phase 4 wire CV	3 phase 4 wire CV	3 phase 4 wire CV

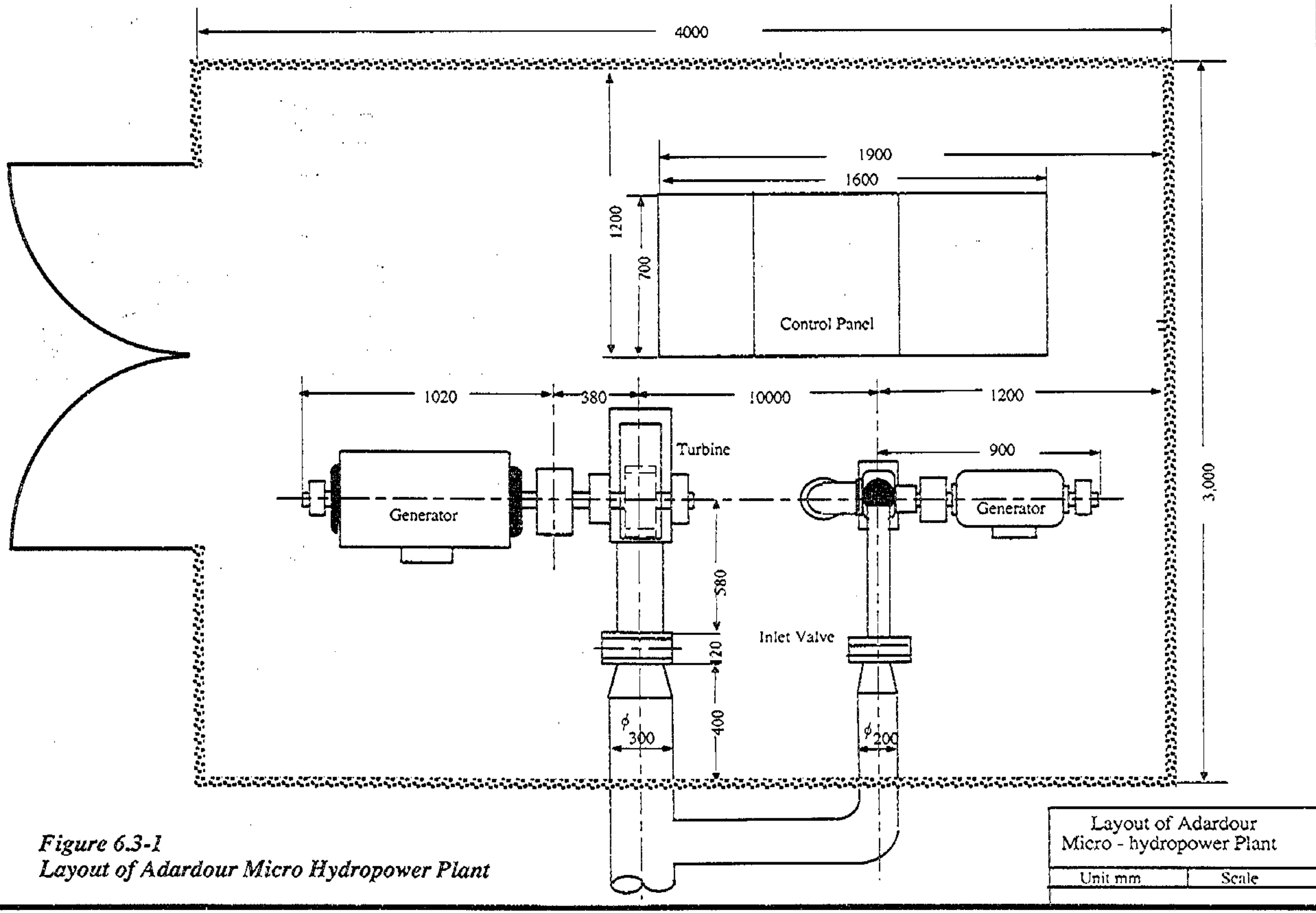


Figure 6.3-1
Layout of Adardour Micro Hydropower Plant

Layout of Adardour Micro - hydropower Plant	
Unit mm	Scale

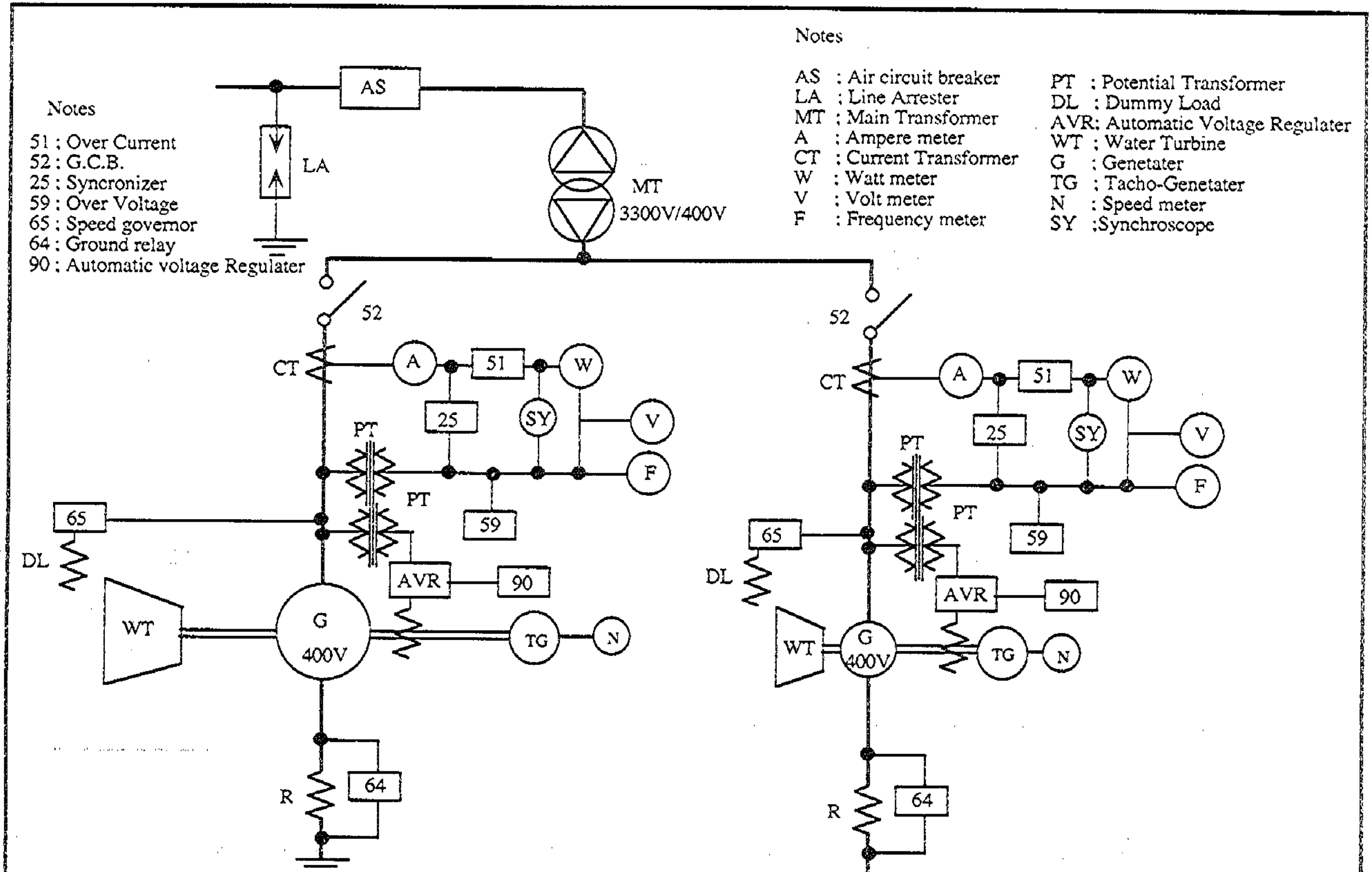
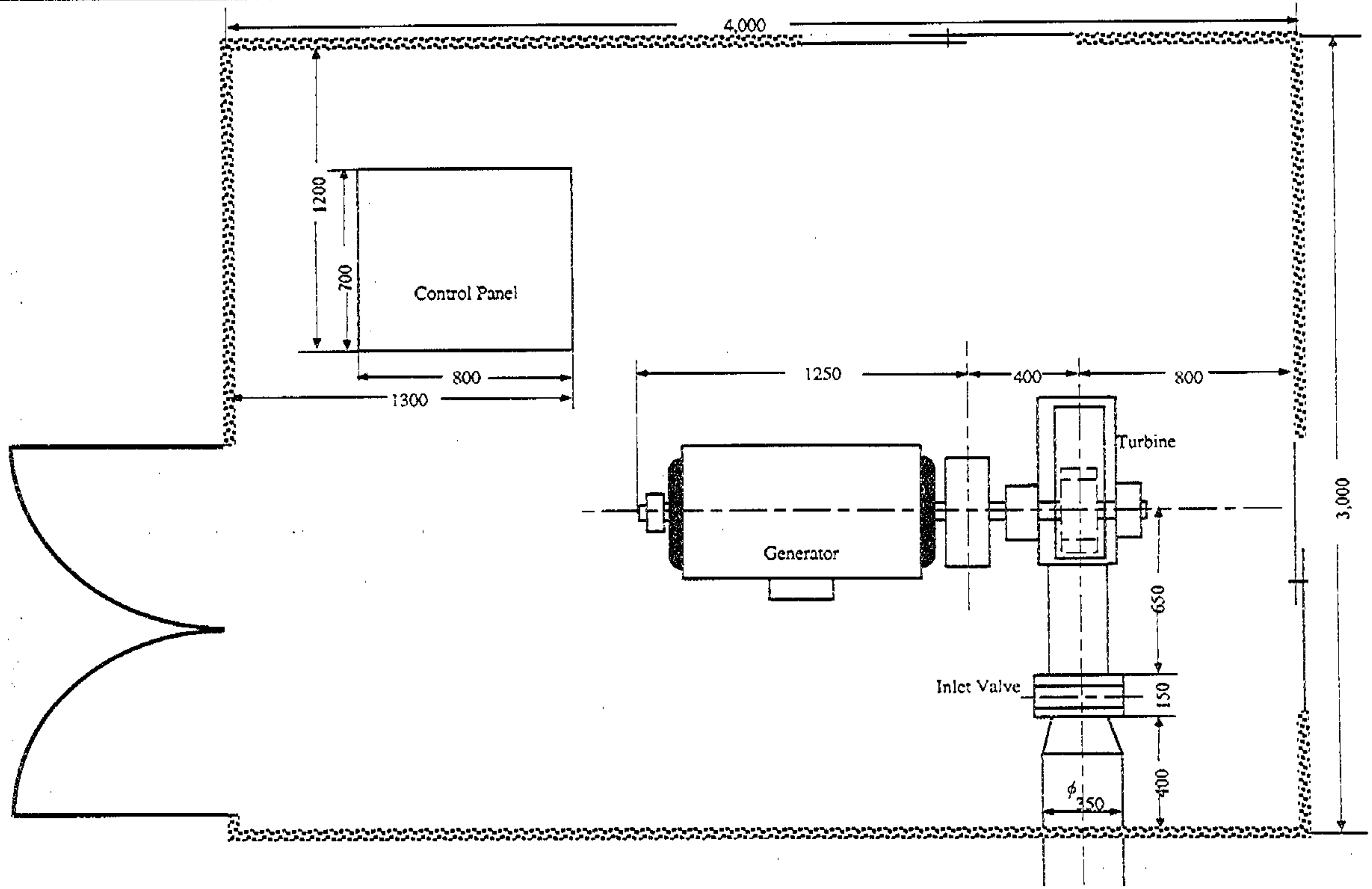


Figure 6.3-2
Single Diagram for Adardour Micro Hydropower System



Layout of Arg
Micro - hydropower Plant

Unit mm	Scale

Figure 6.3-3
Layout of Arg Micro Hydropower Plant

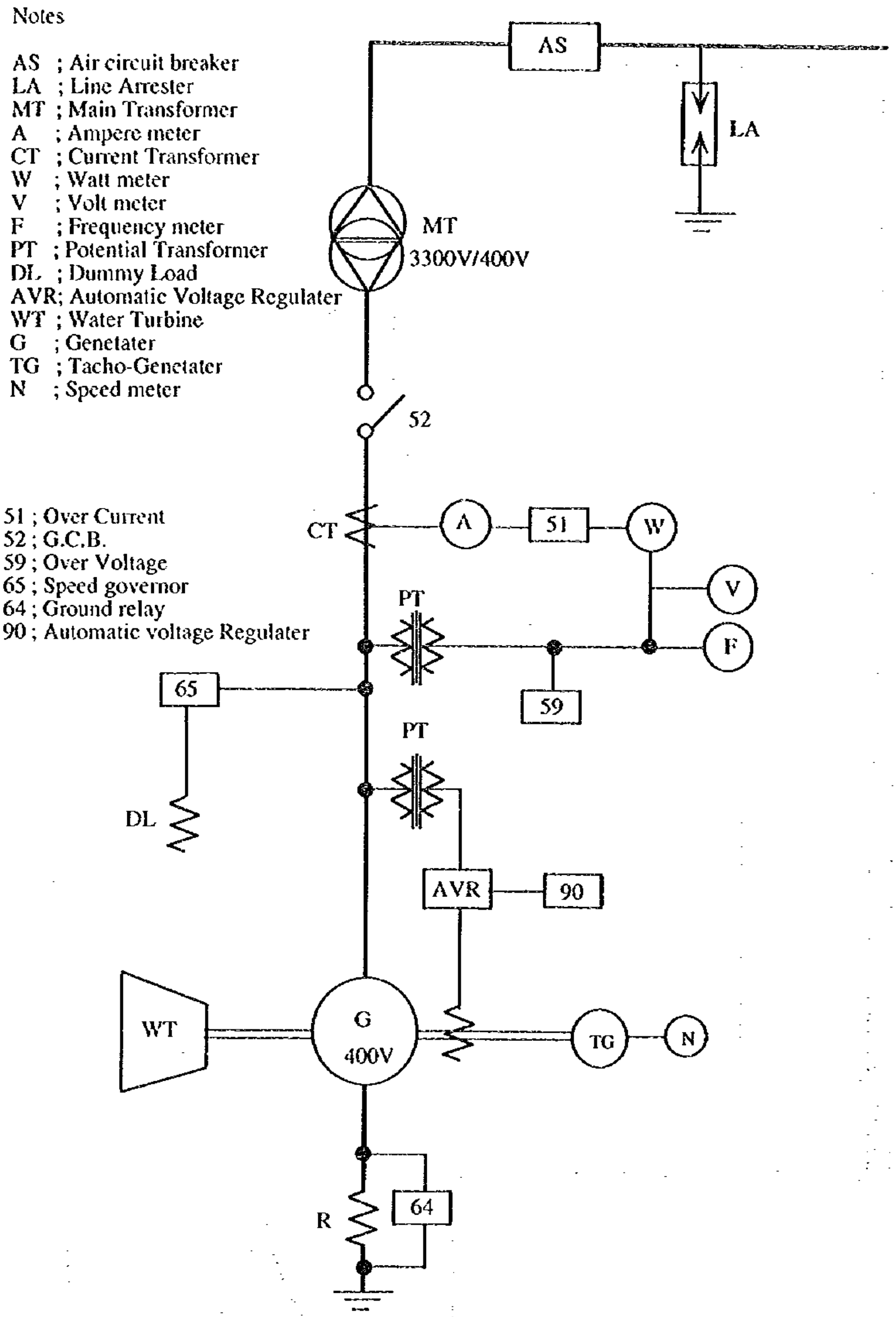


Figure 6.3-4
Single Diagram for Arg Micro Hydropower System

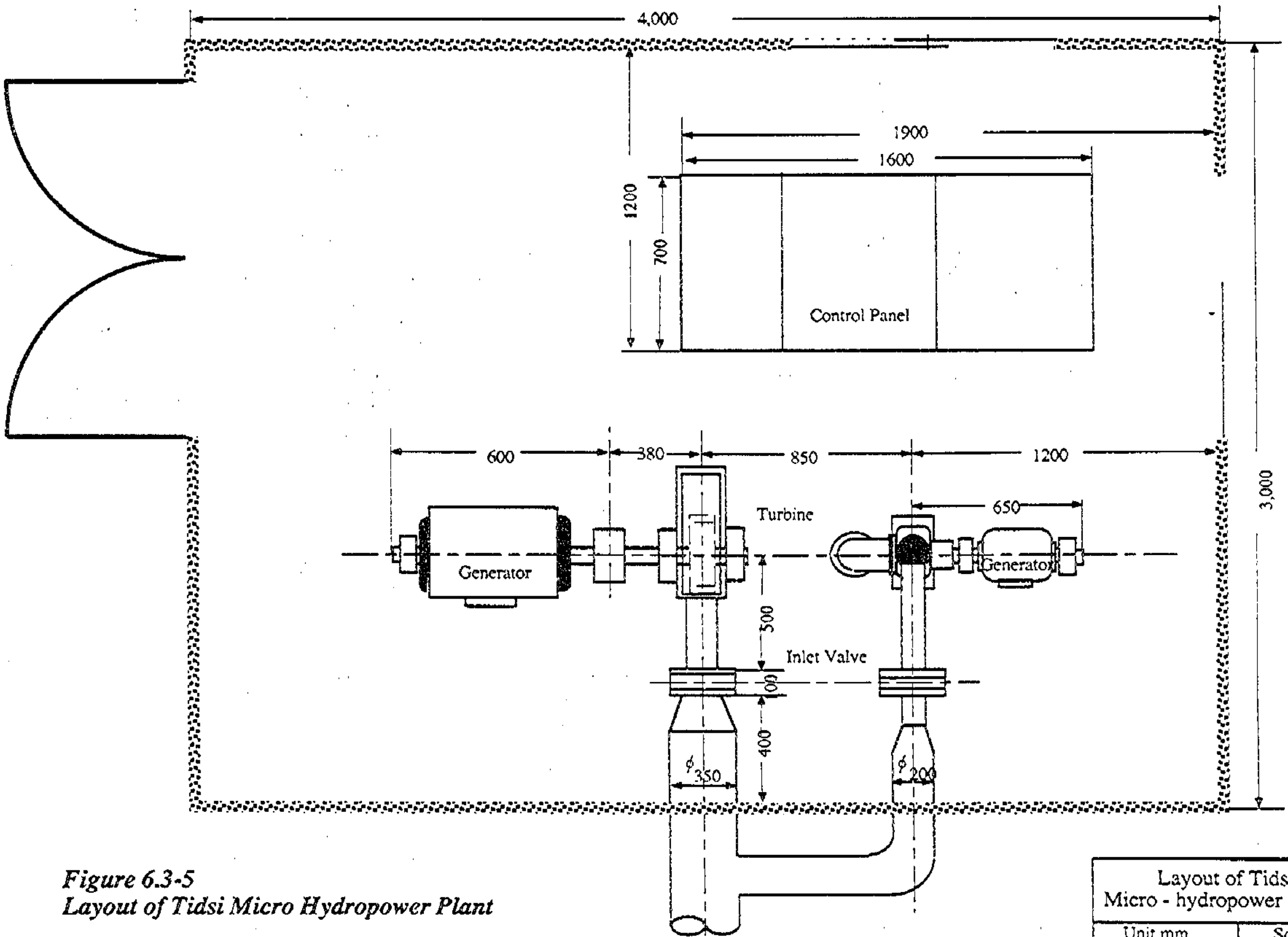


Figure 6.3-5
Layout of Tidsi Micro Hydropower Plant

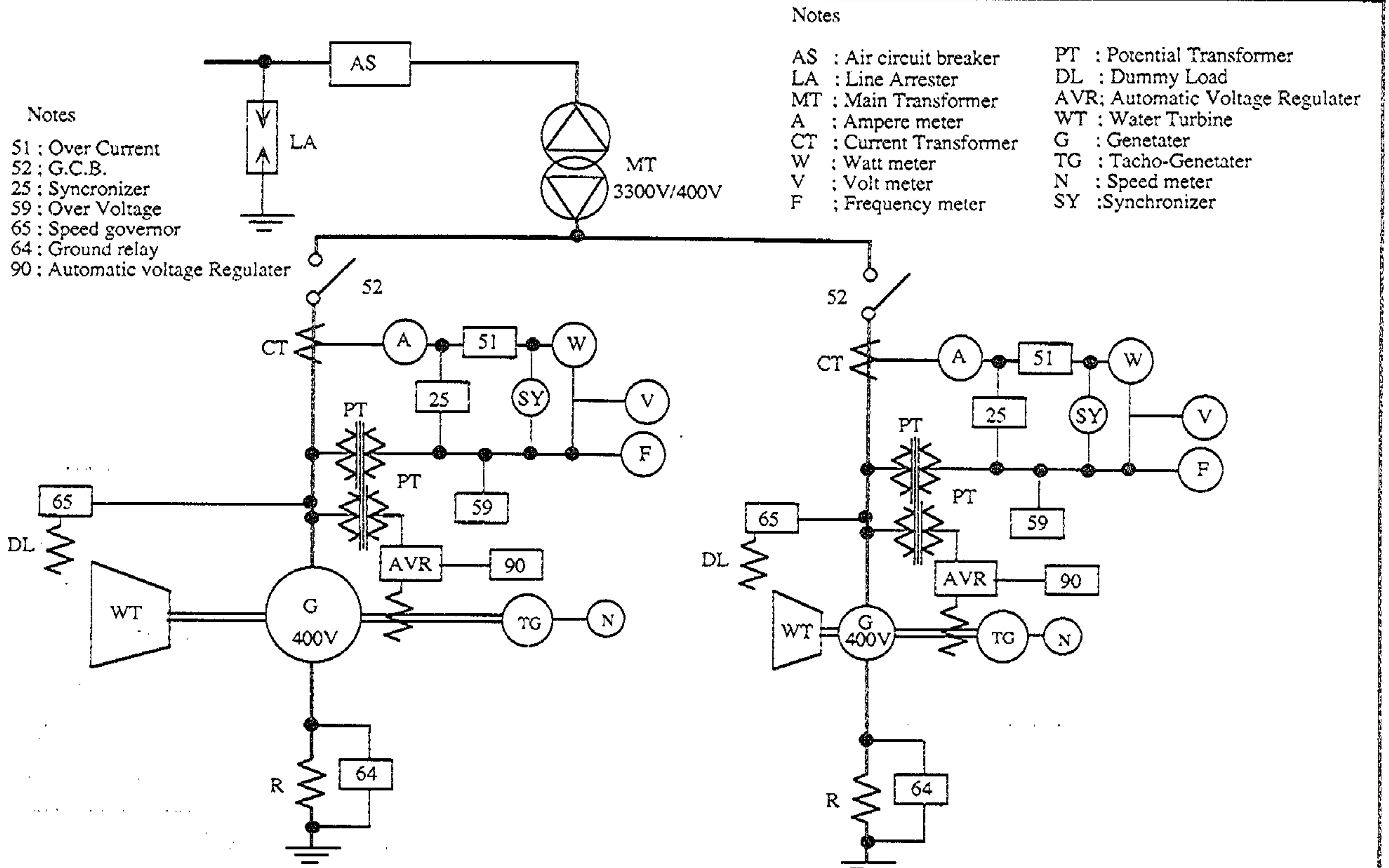


Figure 6.3-6
Single Diagram for Tidsi Micro Hydropower System

VOL. II CHAPTER 7

PROJECT IMPLEMENTATION SCHEDULE

CHAPTER 7 PROJECT IMPLEMENTATION SCHEDULE

7.1 Implementation Schedule

(1) Overall Schedule

As discussed in Section 5.7 of Volume 1, the overall Master Plan will be carried out in two phases. Of this, the subject 3 priority micro-hydropower schemes are to be implemented in Phase I. A description of salient scheme features is given in Table 7.1-1.

Table 7.1-1 Scheme Description

Specification \ Power station	Units	Adardour	Arg	Tidsi
Cercle	--	Amizmiz	Asni	Ait ourir
Commune rurale	--	Anougal	Asni	Tighdouine
Douar	--	Adardour	Arg and others	Tidsi and Afra
River	--	Amizmiz	Rheraya	Zat
Tributary	--	Anougal	Imenane	Afoughal
Catchment area	km^2	23.0	48.0	24.0
Installed output	kW	26.0	30.0	15.0
Maximum discharge	m^3/s	0.11	0.18	0.15
Intake water level	m	1,769.0	1,574.0	1,724.5
Turbine center level	m	1,719.0	1,534.5	1,706.0
Gross head	m	38.0	26.5	16.0
Water way				
Type	-	Open channel	Open channel	Open channel
Length	m	685.0	1,175.0	750.0
Penstock				
Material	-	Steel pipe	Steel pipe	Steel pipe
Dia x Length	m	0.3 x 76.4	0.35 x 84.0	0.35 x 33.8
Power generation				
Turbine (No. 1)	-	Closs flow	Closs flow	Closs flow
(No. 2)	-	Pump reversal	-	Pump reversal

(2) Construction Schedule

The controlling factor in the construction process for the schemes will be headrace canal construction. All headraces are to be open canal structures made of masonry and concrete. As this work is to be performed manually, headrace canal construction constitutes the critical path for the overall implementation of the schemes. Accordingly, headrace construction is to be commenced at the same time for all 3 schemes, and a work execution setup devised to ensure completion within the designated implementation period for the schemes. In the case of structures other than headrace canal, i.e. intake structures, head tanks, penstocks, and power houses, these are to be constructed simultaneous to the aforementioned headrace canal construction, with careful attention to safety measures required in mountainous areas (prevention of rockfall) and elicitation of the cooperation of residents in the construction areas.

Transport of construction material and equipment into the site areas will be via the existing access roads. However, since final approach to the sites from the main highways will be via mountain roads (piste) which are single lane and unsurfaced, a thorough program of access road maintenance and repair must be in effect throughout the construction period. The envisioned implementation schedule for the micro-hydropower schemes, based on the above criteria, is indicated in Table 7.1-2.

Procurement of construction material and equipment is planned as follows based on the Study findings:

1) Construction equipment and facilities are to be procured in Morocco.
2) Construction materials including cement, steel, lumber, etc. are to be procured in principle in Morocco. However, penstock pipe would be imported material procured locally.
3) Lubricants and fuel can be procured from merchants in or near the site areas.

7.2 Implementation Plan

7.2.1 Access Road

The status of the road network centering on Marrakech is as indicated in Figure 2.1-6 in Volume 1. First class trunk highway would be utilized for access to Marrakech from Rabat via Casablanca. The distance from Rabat to Casablanca is 99 km, and that from Casablanca to Marrakech is 237 km. Use of the Rabat~Marrakech bypass results in a distance of 324 km. However, attention must be given to a vehicle height limitation of 4.5 m in order to pass under a number of freeway and railway underpasses along the route. Below, the road conditions from Marrakech to the individual sites is discussed.

(1) Adardour Site

A second class trunk road would be used from Marrakech to Amizmiz (S-507, L = 49 km). The Adardour site would then be accessed from Amizmiz by mountain road (piste) (20 km). The upper basin river crossing along this segment becomes impassable for several days at a time in the rainy season due to flooding. Road width is a narrow 2.0~3.0 m, and alignment profile and curve turning radii pose severe driving conditions. Accordingly, vehicle use would be limited to small trucks for transport (under 1 ton). During the actual construction period as well for effective O&M activities after scheme completion, it is necessary that a thorough program of road maintenance be pursued under the Project including shoulder reinforcement, surfacing, cut-out construction (so vehicles traveling in opposite directions can pass each other under single lane conditions), etc.

Marrakech~Amizmiz:	Second class highway (S-507), 54 km length
Amizmiz~Adardour:	Mountain road, 20 km length

(2) Arg Site

A second class trunk road would be used from Marrakech to Asni (S-501, L = 47 km). The Adardour site would then be accessed from Asni by mountain road (piste) (12 km). The upper basin segment of the piste features several stream crossings which become impassable for several days at a time in the rainy season due to road shoulder collapse as a result of erosion. Road width is a narrow 2.0~3.0 m, and alignment profile and curve turning radii pose severe driving conditions compared to trunk road. Accordingly, vehicle use would be limited to small trucks for transport (under 1 ton). During the actual construction period as well for effective O&M activities after scheme completion, it is necessary that a thorough program of road maintenance be pursued under the Project including shoulder reinforcement, surfacing, cut-out construction (so vehicles traveling in opposite directions can pass each other under single lane conditions), etc.

Marrakech~Asni:	Second class highway (S-501), 47 km length
Asni~Arg:	Mountain road, 12 km length

(3) Tidsi Site

A first class trunk road would be used from Marrakech to Tadert (P-31, L = 95 km). The Tidsi site would then be accessed from Tadert by mountain road (piste) (7 km). As in the case of the other two schemes, this piste segment becomes impassable for several days at a time in the rainy season due to shoulder collapse, surface scouring, etc. Road width is a narrow 2.0~3.0 m, and alignment profile and curve turning radii pose severe driving conditions in comparison to trunk road. Accordingly, vehicle use would be limited to small trucks for transport (under 1 ton). During the actual construction period as well for effective O&M activities after scheme completion, it is necessary that a thorough program of road maintenance be pursued under the Project including shoulder reinforcement, surfacing, cut-out construction (so vehicles traveling in opposite directions can pass each other under single lane conditions), etc.

Marrakech~Tadert:	First class highway (P-31), 95 km length
Tadert~Tidsi:	Mountain road, 7 km length

7.2.2 Civil Construction

(1) General

In the case of all three schemes under the Project, civil works construction extends continuously from the intake facility, along the headrace to the power house, and on to the tailrace outlet. Since the content of these works are linked, it is necessary that overall construction approach be such as to maximize efficiency. Although all structures represent exposed construction, scale is small and sites are located in rugged terrain which would make mechanized construction works impractical. Accordingly, works would be basically done manually with the support of very general construction equipment.

Construction materials including cement, rebar, steel pipe and gates would all be transported to the sites from Marrakech. Concrete aggregate, and masonry stone would be quarried from selected river deposits at the site. Where sufficient such materials are not available at the immediate site, these would be quarried and transported from downstream areas. Unit weight of steel structural material such as pipe, gates, screens, etc. should be as light as possible in consideration of the need for transport along mountain roads, and unloading and installation conditions at the site. Maximum unit weight of turbine and generator equipment components, the heaviest pieces of equipment to be transported to the sites, is to be under 300 kg.

Construction machinery would mainly comprise general construction equipment (drill, compressor, diesel generator, welding equipment, etc.), portable mixer and transport vehicles.

Although all three sites feature existing access roads, these are mountain roads of limited capacity and safety for construction purposes. Accordingly, road improvement (shoulder reinforcement, cut-out construction, etc.) is necessary as a prelude to the start of main construction works at the sites.

Movement of material and equipment to the various construction sites within the scheme would be by donkey or manually carried along existing paths (1.0~1.5 m) to be widened and improved in preparation for the works.

(2) Canal and Power House

1) Adardour

Intake weir, intake structure and desilting basin would be constructed by the coffer dam method, starting with left bank construction. The right bank intake weir segment would be constructed upon completion of the left bank segment, by coffer dam diversion of river discharge to the left bank intake.

The total headrace length of 685 m including diversion facilities and head tank must be completed within a 16 month period. Accordingly, the headrace alignment is to be divided into 3 construction segments, to be implemented simultaneously.

Penstock and power house construction will topographically comprise higher elevation and lower elevation works. In order to prevent danger of rock fall, penstock excavation would be carried out first, following which power house construction would then be commenced. In cases where penstock pipe installation and power house construction works are done at the same time, it will be necessary to carry out thorough safety measures in the form of rock fall prevention works.

Transport to site and installation of turbine and generator equipment would be done upon completion of the power house construction works. Penstock pipe (unit sections with flanges), gates and screens would be finished products ex-factory for subsequent transport to each site and installation thereat.

2) Arg

Intake weir, intake structure and desilting basin would be constructed by the coffer dam method, starting with left bank construction. The right bank intake weir segment would be constructed upon completion of the left bank segment, by coffer dam diversion of river discharge to the left bank intake.

The total headrace length of 1,175 m including diversion facilities and head tank must be completed within a 16 month period. Since the entire length of the planned headrace will comprise rehabilitation of existing irrigation canal, construction conditions are relatively facilitated in comparison to other sites. However, due to the length of the said canal, the headrace alignment is to be divided into 2~3 construction segments, to be implemented simultaneously.

Penstock and power house construction will topographically comprise higher elevation and lower elevation works. In order to prevent danger of rock fall, penstock excavation would be carried out first, following which power house construction would then be commenced. In cases where penstock pipe installation and power house construction works are done at the same time, it will be necessary to carry out thorough safety measures in the form of rock fall prevention works.

Transport to site and installation of turbine and generator equipment would be done upon completion of the power house construction works. Penstock pipe (unit sections with flanges), gates and screens would be finished products ex-factory for subsequent transport to each site and installation thereat.

3) Tidsi

Intake weir, intake structure and desilting basin would be constructed by the coffer dam method, starting with right bank construction. The left bank intake weir segment would be constructed upon completion of the right bank segment, by coffer dam diversion of river discharge to the right bank intake.

The total headrace length of 750 m including diversion facilities and head tank must be completed within a 16 month period. Since the entire length of the planned headrace will comprise new construction (concrete open canal), construction schedule management and quality control become important factors. Fortunately, there is no forest along the canal route and access is relatively easy, and in this light the headrace alignment is to be divided into 2~3 construction segments to be implemented simultaneously.

Penstock and power house construction will topographically comprise higher elevation and lower elevation works. In order to prevent danger of rock fall, penstock excavation would be carried out first, following which power house construction would then be commenced. In cases where penstock pipe installation and power house construction works are done at the same time, it will be necessary to carry out thorough safety measures in the form of rock fall prevention works.

Transport to site and installation of turbine and generator equipment would be done upon completion of the power house construction works. Penstock pipe (unit sections with flanges), gates and screens would be finished products ex-factory for subsequent transport to each site and installation thereat.

7.2.3 Generating Equipment

(1) Procurement and Transport of Equipment

Turbine and generator equipment are to be procured from off-shore. Modern port facilities are available at Casablanca (nearest port), and there will consequently be no problems in terms of equipment off-loading and handling.

Off-loaded equipment would then be transported via Marrakech (240 miles to the south southwest) to the sites. Conditions for transport of the said equipment to each site from Marrakech by truck are as follows:

① In the case Adardour, a surfaced road exists up to Amizmiz. From this point, access to the site is by a 20 km mountain road. There are existing cut-outs along the road to allow passing of small vehicles from opposite directions; however, these are not surfaced. Given these conditions, it is necessary to limit transport weight to under 300 kg.

② In the case of Arg, access to the site from Asni is by 12 km mountain road. Road conditions are the same as for Adardour, and in this light transport weight is to be limited to under 300 kg.

③ In the case of Tidsi, access to Tadert is by 95 km trunk highway from Marrakech. From Tadert, access to the site is by 7 km mountain road. Since the 2 km segment after Afra is considered to pose problems for vehicle transport of materials, transport weight is to be limited to under 200 kg.

(2) Receiving of Equipment at the Sites and Storage

As no equipment receiving facilities are to be constructed at the sites, this work will all have to be done manually. The necessary equipment for installation including chain blocks, roller equipment, jacks, lifting forks, etc. must by carefully readied and stored at the site ahead of time. Sheltered storage at the site will be limited to the power house structure, and upon completion of the same it is recommended that equipment be stored therein.

(3) Generating Equipment Installation Work

Equipment is to be installed upon completion of foundation works. Heavy pieces of equipment would be installed using the overhead hook and fork lift equipment put in place in advanced. Movement of equipment within the power house structure itself would be by roller, or done manually. Equipment would be installed in reference to the standard alignment center for penstock and turbine. Governor and control panel would be installed following completion of turbine and generator installation. Since governor control is by the "dummy load control" method, resistance device will be placed inside the tailrace itself making it necessary to give ample care to insulation protection.

(4) Installation Works for Outdoor Equipment

Outdoor equipment at the power house will comprise the main transformer, transmission switch gear and lightning grounding equipment. Although it is a general practice to posit the main transformer on a ground foundation, in the case of the subject schemes the said equipment is to be pole mounted in the interests of safety and given the topographical conditions at the sites. Since the total weight of the main transformer is 600 kg, it is to be mounted on a frame supported by two poles.

The operating panel for transmission switch gear is to be installed near the ground surface to allow for control from the ground. This equipment, however, would be housed in a protective locker to prevent access by unauthorized persons.

7.2.4 Transmission and Distribution Facilities

Given constraints on material procurement and transport, power poles for transmission and distribution are to be wooden in principle. Pole height from the ground would be 8.0~10.5 m, with imbedded depth of at least 1.6 m. Transmission voltage is to be 3,300 V and distribution voltage is to be 380 V / 220 V. Accordingly, pole mounted transformer for voltage adjustment are to be installed on the designated power poles. In mounting the same, ample care must be given to durability against the more severe wind and rain conditions prevailing in mountain areas.

In addition to carrying out connection works to each user household and public facility, electricity meters are also to be installed to gauge power consumption. Specifically in the case of households, the following are to be done: electricity meter installation, house wiring and installation of 6 electrical outlets for lighting purposes.

Table 7.1-2 Micro-hydropower Scheme Implementation Schedule